US Army Corps
of Engineers
Hydrologic Engineering Center

Floodway Determination Using Computer Program HEC-2

January 1988

TD-5

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	January 1988	Training Document No. 5

4. TITLE AND SUBTITLE
Floodway Determination Using Computer Program HEC-2

5. FUNDING NUMBERS

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8. PERFORMING ORGANIZATION REPORT NUMBER
TD-5

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)
N/A

10. SPONSORING / MONITORING AGENCY REPORT NUMBER
N/A

11. SUPPLEMENTARY NOTES
N/A

12a. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for Public Release. Distribution of this document is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

This document illustrates how computer program HEC-2 can be used to develop a designated floodway as required for Type 15 flood insurance studies. It was assumed that the reader has knowledge of the basic input requirements. The computer procedure for delineating a floodway includes: (a) the development of water surface profiles under natural conditions (i.e., prior to encroachment), and (b) the delineation of a designated floodway that meets certain requirements. The procedure in the HEC-2 encroachment routines allows a program user to make the preliminary estimate of a designated floodway in one operation of the program. Additional computer runs may be made to improve the acceptability of the floodway. The computerized procedure was discussed and illustrated in an example problem. Appendices contain a hand calculation example to illustrate the procedure, and first and second trials of a simple problem using the HEC-2 program. The User's Manual, available from the Hydrologic Engineering Center, provides the input specifications.

14. SUBJECT TERMS
computer models, floodways, hydrology, flood plains, channels, hydraulics, flow profiles, computer programs, data processing, flood routing, training, model studies

15. NUMBER OF PAGES
128

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED	UNLIMITED

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FLOODWAY DETERMINATION USING COMPUTER PROGRAM HEC-2

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1. INTRODUCTION

1.1 PURPOSE

This document presents how computer program HEC-2 can be used to develop a floodway based on the Federal Emergency Management Agency (FEMA), Flood Insurance Administration (FIA) Guidelines and Specifications (FEMA, Sep.1985). It is assumed that the reader has a knowledge of the basic program capabilities and input requirements described in the HEC-2 Users Manual (HEC, 1982).

1.2 OVERVIEW

Terms frequently used in floodway analysis are defined in Chapter 2. The regulatory basis for the floodway concept is presented in Chapter 3. The encroachment options in HEC-2 were developed to provide a convenient method for performing the hydraulic analysis, based on the floodway concept.

The approach followed by the Corps of Engineers for flood plain management is presented in a paper entitled "The Regulatory Floodway" (USACE, August 1972). A procedure for delineating a floodway is given in a paper titled "General Guidelines for Development of Floodway Data" (USACE, March 1972). Chapter 3 presents the procedures outlined in those documents and Chapter 4 provides an example computation to illustrate the HEC-2 options and procedure. The example is simple to provide an illustration of the basic input, output, and analysis.

Some of the basic floodway application problems, including model assumptions, are reviewed in the Chapter 5. Also, several difficult application situations are defined and discussed. The material in Chapters 6 through 10 provides a review of typical problems associated with each situation and suggests modeling approaches.

1.3 ACKNOWLEDGMENTS

The original version of this document was written by Vern Bonner in 1974. Funding was provided by the Flood Plain Management Branch, Office, Chief of Engineers. Material included in the document was based on lecture notes and handouts developed by Bill S. Eichert and John Peters. Computer program HEC-2 was developed by Mr. Eichert and the encroachment routines were added by him in 1971. Additional encroachment Methods 5 and 6 were added in 1978.

This revision was made by Vern Bonner to include material developed during a contract study of the floodway concept, performed in 1987 for FEMA (HEC, 1987). Portions of that study report are included in Chapters 2, 3 and 5 through 10 of this

document. The study was performed under the management of Vern Bonner, and he wrote the report. Mr. Richard Hayes provided data collection and technical analysis for the study. Dr. Robert MacArthur, Simons, Li & Associates, Inc. reviewed floodway applications on alluvial streams and wrote the draft of that chapter. Technical support and reviews were provided by John Peters, Michael Gee, and Alfredo Montalvo. Ms. Lynne Stevenson provided editorial reviews. Mr. Bill S. Eichert was Director, HEC, during the conduct of the study and provided technical review and guidance.

The example computation in this document reflects the modifications made in the HEC-2 program since the original document was written. The computer runs were performed with the 1988 PC Version of HEC-2, which was still under development by Randy Hills when this document was revised. Therefore, there may be some slight differences in your program output, when compared to the examples in this document.

2. DEFINITIONS

The following are definitions of terms frequently used in floodway analysis. Where there is a source for the definition, it is given. Otherwise, the definition is offered based on the general consensus of the HEC staff and others. The definitions are grouped by type so that they can be more easily compared.

2.1 CLASSIFICATIONS OF OPEN-CHANNEL FLOW (Chow, 1959)

STEADY FLOW - Depth of flow does not change with time.

SUBCRITICAL FLOW - State of flow where the gravitational forces are more pronounced than the inertial forces. The flow has a low velocity and is often described as tranquil and streaming.

SUPERCRITICAL FLOW - State of flow where the inertial forces become dominant. The flow has a high velocity and is usually described as rapid, shooting, and torrential.

UNSTEADY FLOW - Depth of flow changes with time.

UNIFORM FLOW - Depth is constant over channel length. (Prismatic channel required)

VARIED FLOW - Depth of flow changes along the channel length.

GRADUALLY VARIED FLOW - Depth changes gradually over the channel length.

RAPIDLY VARIED FLOW - Depth changes abruptly over a comparatively short distance.

2.2 HYDRAULIC TERMS (Chow, 1959)

CONVEYANCE - A measure of the carrying capacity of the channel section. Flow is directly proportional to conveyance. From Manning's equation, the proportional factor is the square root of the energy slope.

CRITICAL DEPTH - The depth at which the specific energy for a given flow is at a minimum. The depth dividing supercritical (below critical depth) and subcritical (above critical depth) flow.

FROUDE NUMBER - A dimensionless number representing the ratio of inertial forces to gravitational forces. A Froude number of 1.0 indicates critical flow. A Froude number less than one indicates subcritical flow and greater than one indicates supercritical flow.

HYDRAULIC DEPTH - An average depth computed by dividing the cross-sectional flow area by the width of the free water surface (top width). It is the flow depth for rectangular channels.

SPECIFIC ENERGY - The energy in a channel section per unit weight of water, measured with respect to the channel bottom. Equal to the depth of water plus the velocity head.

VELOCITY HEAD - The kinetic energy term (alpha $V^2/2g$) in the total energy of flow. The energy (or Coriolis) coefficient (alpha) is used to adjust for the distribution of velocity in the cross section.

2.3 FLOOD TERMS

BASE FLOOD - The flood discharge used to define floodways for flood insurace studies. The base flood has a 1-percent chance of being exceeded in any given year.

EXCEEDANCE FREQUENCY - The percentage of values that exceed a specified magnitude, 100 times exceedance probability (WRC, 1982).

FLOOD - A general and temporary condition of (1) partial or complete inundation of normally dry land areas from the overflow of inland and/or tidal waters and/or (2) the unusual accumulation of waters from any source (FEMA, 1986).

RECURRENCE INTERVAL - The average interval of time (eg., 100-years) in which a flood of a given size is exceeded as an annual maximum (WRC, 1982).

1-PERCENT CHANCE FLOOD - (an exceedance frequency) The flood that has a 1-percent chance of being exceeded in any given year; equivalent to the 100-year flood.

100-YEAR FLOOD - (a recurrence interval) The flood that is exceeded once in 100 years on the average; equivalent to the 1-percent chance flood.

2.4 FLOOD HAZARD TERMS

FLOOD HAZARD - The potential for inundation which involves the risk to life, health, property, and natural flood plain values (FEMA, 1986).

FLOOD HAZARD RATING (WRC, 1969)

PHYSICAL HAZARD:	LOW	MEDIUM	HIGH
1% Flood Depth:	< 1 ft	1-3 ft	> 3 ft
Flood Rise Time:	> 24 hrs	12-24 hrs	< 12 hrs
Flood Velocity:	< 1 fps	1-3 fps	> 3 fps
Flood Duration:	< 6 hrs	6-24 hrs	> 24 hrs
Site Access:	good	fair	poor

HAZARDOUS DEPTH - Based on WRC hazard criteria, a depth greater than three feet would be considered as hazardous. However, this does not consider velocity which would compound the hazard.

HAZARDOUS VELOCITY - Based on WRC hazard criteria, a velocity greater than three feet per second would be considered a hazard.

HIGH VELOCITY (subcritical flow) - May be considered equivalent to Hazardous Velocity; however, for this report it is considered to be greater than 5 feet per second. The definition could also be dependent on bed material scour, which might set the velocity between 2 to 6 feet per second for many channel materials (USACE, 1970).

LOW VELOCITY - Based on WRC criteria for a hazard, a velocity of less than one foot per second would be considered a low velocity. General engineering usage would assume a higher value.

SHALLOW FLOODING - For the purpose of the NFIP, shallow flooding conditions are defined as flooding that is limited to three feet or less in depth where no defined channel exists (FEMA, Sep. 1985). A flood hazard distinction is made between depths less than one foot and depths between one and three feet.

2.5 FLOOD PLAIN TERMS

FLOOD PLAIN - The lowland and relatively flat areas adjoining inland and coastal waters, and those other areas subject to flooding (FEMA, 1986).

FLOOD PLAIN VALUE - Those natural and beneficial attributes associated with the relatively undisturbed state of the flood plain and include values primarily associated with water, living, and cultural resources (FEMA, 1986).

FLOODWAY - The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. Normally the base flood is defined as the 1-percent chance flood and the designated height is one foot above the pre-floodway condition (FEMA, Sep. 1985).

FLOODWAY FRINGE - The area between the floodway and the base flood plain boundaries. The fringe is that portion of the flood plain that could be completely obstructed without increasing the water surface elevation of the 1-perecent chance flood by more than a designated height (usually one foot) (FEMA, Sep. 1985).

WIDE FLOOD PLAIN - Generally a flat flood plain where there is a potentially large increase in the extent in lateral flooding from a small (e.g., one foot) change in water surface elevation.

2.6 STREAM CLASSIFICATION TERMS

ALLUVIAL STREAM - A stream that has formed its channel by the process of aggradation. The sediment in the stream is similar to the material in the bed and banks (ASCE, 1968).

BRAIDED RIVER - A river for which the channel is extremely wide and shallow and the flow passes through a number of small interlaced channels separated by bars or shoals. Slopes are normally steep. There is little or no erosion of the main banks. The channel as a whole does not meander, although local meandering in minor channels generally occurs (ASCE, 1968).

BRAIDED STREAM - Elements of a network of connecting stream channels on an alluvial fan or plain or a delta (ASCE, 1968).

LOW GRADIENT STREAM - A stream with a mild slope for which flow is subcritical. Sometimes used to refer to very mild slopes with low velocities, e.g., less than five feet per second.

MILD SLOPE - A slope less than the critical slope for a particular discharge; the slope for which the normal depth of flow is greater than critical depth and the velocity is less than critical velocity (ASCE, 1968).

PERCHED STREAM - A stream where the overbank area is lower than the immediate river bank elevation. When the river overflows the banks, the water tends to move laterally away from the main channel.

STEEP SLOPE - A slope greater than the critical slope for a particular discharge; the slope for which the normal depth of flow is less than critical depth and the velocity is greater than critical velocity (ASCE, 1968).

3. REGULATORY BASIS FOR FLOODWAY

3.1 FLOOD INSURANCE BACKGROUND

"The National Flood Insurance Program (NFIP) was established by the National Flood Insurance Act of 1968 and further defined by the Flood Disaster Protection Act of 1973. The 1968 Act provided for the availability of flood insurance within communities that were willing to adopt flood plain management programs to mitigate future flood losses. The Act also required the identification of all flood plain areas within the United States and the establishment of flood-risk zones within those areas."

"A vital step toward meeting these goals is the conduct of Flood Insurance Studies and restudies for flood-prone communities. These studies provide communities with sufficient technical information to enable them to adopt the flood plain management measures required for participation in the NFIP. They also develop the flood risk information necessary to establish actuarial flood insurance premiums." (FEMA, Sep. 1985, p1-1)

The Flood Insurance Study (FIS) provides the basis for the determination of base flood elevations, risk zones, insurance rates, community controls and management of land uses and thus promotes the NFIP objectives (FEMA, March 1986). Based on consultation with the community, the scope of a study is established by considering areas which are developed or are likely to be developed within the next five years. The flood hazard areas are determined on the basis of historic flood experience.

The main product of the FIS is a Flood Insurance Rate Map (FIRM) which depicts 100- and 500-year flood boundaries, flood insurance rate zones, and base flood elevations (FEMA, March 1986). Areas subject to inundation by the base (100-year) flood are labeled as Special Flood Hazard Areas (SFHAs). The zones assigned depend on whether approximate or detailed methods are used and on the depth and type of flooding (FEMA, Sep. 1985). Areas between the 100- and 500-year flood boundaries are termed areas of Moderate Flood Hazard. The remaining areas, outside the 500-year boundary, are termed areas of Minimal Flood Hazard.

An accompanying FIS report is usually published with a FIRM. The boundaries of 100- and 500-year flood plains and the floodway may be shown on a separate Flood Boundary and Floodway Map (FBFM), published as part of the FIS report or on the FIRM. The FIRM and FBFM are used by the community for enacting and enforcing flood plain management regulation ordinances and ensuring sound construction practices in flood hazard areas (FEMA, March 1986).

3.2 FLOODWAY CONCEPT

A floodway is "the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water-surface elevation more than a designated height" (FEMA, Sep. 1985). The floodway serves to conserve an unobstructed area for the passage of floodwaters while providing for the appropriate use of adjacent lands based on knowledgeable awareness of the flood hazard (USACE, August 1972).

The following guidelines provide a uniform approach to floodway determinations. They are used to develop the initial floodway, unless the state has more restrictive requirements through legally enforceable statutes. The Flood Insurance guidelines are defined by FEMA in Document No. 37, commonly referred to as "the Guidelines," (FEMA, Sep. 1985).

a. The flood plain and the capacity of the floodway will be based on the discharge of the 1-percent chance flood.

b. The flood plain will be divided into two zones, the floodway and the floodway fringe (see Figure 1).

c. The floodway will be designed to pass the base flood discharge without raising the water surface elevation more than one foot above the water surface elevation for the existing flood plain.

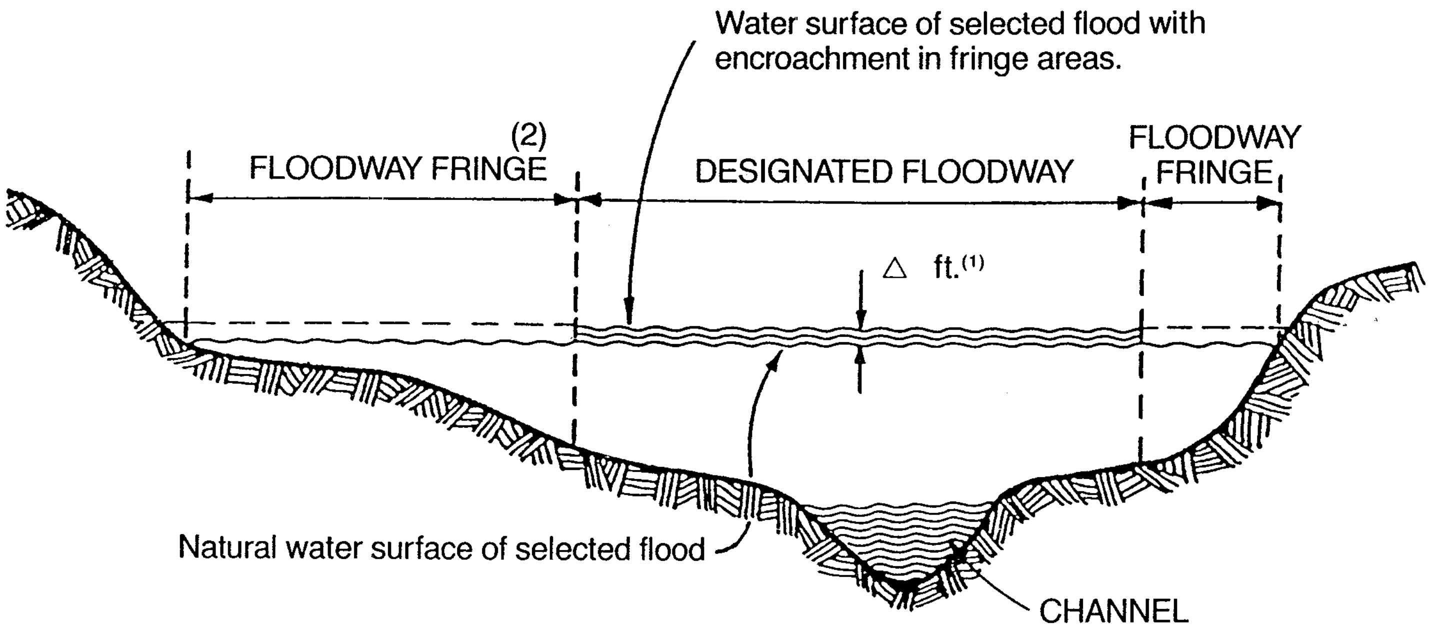
d. The floodway fringe is that portion of the flood plain between the floodway and the limits of the existing 100-year flood plain. For hydraulic computations, it is generally assumed that all conveyance in the floodway fringe area will be eliminated and provide no flow capacity.

e. The floodway will be determined using an equal reduction of overbank conveyance on both sides of the stream. There are provisions for deviation from this requirement as long as the stream is not the boundary between different communities.

f. The hydrology and hydraulics for floodway determination are based on existing conditions. Specific guidelines are provided for consideration of future flood control works.

g. The final floodway is based on input from the local community and coordination with state officials.

While other considerations are suggested in other guidelines (USACE, March 1972, 1978), the FEMA guidelines do not require them. Generally, the special considerations involve concerns about potential increased hazards that could be caused by flood plain encroachments; e.g., increased velocities and loss of valley storage. Some of these added considerations have been adopted by individual states as part of their floodway criteria.



(1) *Maximum of 1 foot or less if so established by State or local regulations.*

(2) *The floodway fringe should normally be considered as the area between the designated floodway limit and the limit of the natural selected flood as long as the encroachment results in only an insignificant increase (1 foot or less) in the water surface elevation of the selected flood.*

Figure 3.1. Floodway Definition Sketch

The concept of a maximum 1 foot rise evolved during the mid-fifties as a realistic limit for a "significant increase"; considering the engineering judgment applied in the hydrologic and hydraulic computations. It provided for development in the flood plain while preserving an essential conveyance zone for flood flow (Goddard, 1978).

3.3 COMPUTATIONAL PROCESS

The computational process is described below, based on the Corps' suggested procedures (USACE, 1978). The discussion and examples refer to the use of the Corps' Water Surface Profile Program HEC-2 (HEC, Sep. 1982), which is frequently used for these studies. However, the reference to HEC-2 does not limit the concepts, procedures, or results to that program because the other available standard step backwater programs are fundamentally similar.

a. Develop and calibrate a model of the study reach. Ideally, the calibration process will reproduce water surface profiles for some historic floods where high water marks and/or gage records are available.

b. Compute water surface profiles for a range of discharges, including the 1- and 0.2-percent chance floods.

c. Analyze water surface profiles for the the study reach and ensure that the model is producing reasonable results.

d. With the calibrated model, compute the base flood water surface profiles for existing and potential floodway conditions. The HEC-2 program has several options for estimating the initial encroachment limits based on reducing equal conveyance, modifying the cross section to eliminate the encroached area, and computing the resulting water surface elevation.

e. Evaluate the results of the computed profiles for the various proposed floodways. The primary criterion is the change in water surface elevation. However, the review should also consider excessive velocities, floodway alignment, and local development and requirements.

f. Based on the review of the preliminary floodway results, refine the floodway definition and repeat the computation process until the evaluation criteria are met.

g. Define and present the floodway to interested agencies and make any additional adjustments necessary. The result is labeled the <u>final</u> floodway.

The computational process provides water surface elevations and locations for encroachment limits at defined cross sections. The computational results must be transferred onto a map and the floodway "filled in." Because information is only available at the cross sections, the actual definition of the floodway requires the interpretive extension of the cross-sectional information to define a line between the sections that will be the floodway. Usually the ground contours and the general flood plain shape are used to assist in the definition of the floodway.

3.4 COMPUTATIONAL ASSUMPTIONS

The computer programs used to compute the water surface profiles are intended for "steady, gradually varied flow" (HEC, Sep. 1982). The definition requires two conditions: (1) that flow remains constant for the time interval under consideration, and (2) the streamlines are practically parallel so that a hydrostatic pressure distribution can be assumed (Chow, 1959). The following outlines the assumptions and how they affect the application of the models used to compute floodways.

a. **Steady flow** does not vary with time, but in reality the flood discharge does. The requirement indicates that the flow remains constant for the time of consideration. This is usually interpreted to apply to the general rainfall and snowmelt floods where discharge changes slowly with time. For those floods, a person standing on the bank of the stream would not see the water depth changing with time.

b. **Gradually varied flow**, as the name implies, has a gradual change in depth along the length of the stream. The assumption of hydrostatic pressure distribution in the section is met for still water. For moving water, the assumption is generally considered reasonable as long as the flow changes are gradual so that the imaginary lines of flow can be considered to be reasonably parallel or gradually changing.

c. Flow is **one-dimensional** with a single water surface elevation across the section. The cross-sectional information provided should only represent an area where the flow is perpendicular to that section and has a horizontal water surface elevation. Velocities in directions other than the direction of flow are not evaluated. Also, any effects due to curvature of the river are ignored.

d. River channels should have a **small slope**, less than 1 in 10. Small slopes are required because the pressure head is represented by the water depth measured vertically. Most floodway studies are performed on streams that meet this requirement.

e. The cross section is a **rigid boundary**; it does not change shape with the flow or during the flood. While this assumption is generally used, many alluvial streams may have considerable change in section shape during a flood that approaches a 1-percent chance event.

A thorough understanding of the assumptions, listed above, is required before an adequate model of a study reach can be developed. There is considerable engineering judgment required in locating and defining the cross-sectional data to best meet the assumptions. Additionally, the selection of energy loss coefficients and the calibration of the model can provide significant differences in computed water surface elevations and floodways. Some of the difficulties in computing floodways are a direct result of applications that fail to meet these assumptions.

3.5 IDEAL FLOODWAY APPLICATION

The application of the floodway has two components; application of the concept and computational application. The applicability of the concept is a primary question to be answered. Given the definition of a floodway, does the current procedure adequately define a floodway?

3.5.1 CONCEPT APPLICATION. Floodway determination is based on an evaluation of the stream's conveyance and an assessment of the impact on the water surface profile. The elimination of the conveyance in the floodway fringe assumes that development will occur along the fringe and that development will essentially eliminate the flow carrying capacity in the fringe area.

The computation process described in Section 3.3 and the computational assumptions described in Section 3.4 are generally applied to floodway determinations. Although different water surface profile programs are used to perform the studies, the programs are based on the same concept and theory. Given that the study reach reasonably conforms to the assumptions described and that the model is developed in a competent manner, the computation process reflects the impact of lost conveyance on the water surface elevation. Also, an estimate of flow depths and velocities, under floodway conditions, are provided.

The computation process does not directly reflect the impact of flood plain encroachment on flood plain (valley) storage, peak discharge, or flood wave travel time. While some assessment can be made on these other impacts, they are not usually a direct consideration of the FEMA water surface profile or floodway computation process.

The question whether the elimination of fringe conveyance reflects the impact of flood plain development is not as easy to answer. In a review of floodway determinations and resulting applications, the computed floodways were found to cause less than the maximum allowable change in water surface elevation (Goddard, 1978). That would indicate that the computed floodway meets the FEMA requirements, but does not necessarily reflect the maximum possible encroachment into the flood plain. Also, the actual development was seldom in a manner that completely blocks the the flow or storage in the fringe. The fringe development was found to block from zero to 100 percent of the area, with an average of around 25 percent in the 56 communities reviewed. That would indicate that the assumption of complete elimination of conveyance may be conservative. However, given that the fringe area could be completely filled, it is reasonable to define the limits based on that assumption.

A second part of the development assumption is that it will occur along the fringe of the flood plain. For many locations, where the terrain slopes down toward the river, the fringe of the flood plain, with its shallow depth, is the most convenient and likely place to develop. However, there are flood plains where the overbank area is very flat, or the overbank slopes downward away from the river, or the overbank area is generally flat with mounds of high ground. For these various terrain situations the most logical development locations are not necessarily along the fringe of the flood plain. While it may be possible to compute the floodway, there remains the question of whether it would reasonably reflect the best areas for development in the flood plain.

3.5.2 COMPUTATION APPLICATION. While there are some difficult problems encountered in floodway determination, there are many situations where the study reach does sufficiently fit the ideal model and the process of defining the flood plain and floodway is a straightforward engineering study. The following is focused on the computational aspects of floodway determination, not on any administrative problems that may arise from adopting the computed floodway.

The ideal flood plain consists of a single, well defined low flow channel in a gently sloping flood plain (see Figure 1). The overbank area generally increases in elevation laterally away from the channel, on either side. The cross sections from the ideal flood plain will contain the flow within the boundary geometry and the entire cross section can be considered to effectively carry the flow. Ineffective flow areas (dead storage zones) are few. There is no loss of flow with increasing discharges; a consistent stage-discharge relationship will result from the computation for a range of discharges.

The channel with a mild slope maintains the flow in the subcritical flow regime. The variations in channel shape and slope along the channel are relatively small and gradual. Tributaries to the study reach are few and minor, and they do not significantly affect the water surface profile beyond the added discharge they provide. There are no downstream conditions (e.g., reservoirs or tidal estuaries) that influence the flood level in the area of interest.

There are few manmade structures affecting the flow in the ideal flood plain. Bridge crossings are generally designed to pass the 100-year flow with minimum impact, ideally with all the flow passing under the bridge under low flow conditions. There are no dams, diversions, or other structures controlling the depth or direction of flow in the stream.

While the stream may be alluvial, velocities are relatively low and the assumption of rigid flow boundary is acceptable. This condition may be the most difficult to determine and requires considerable judgment and experience.

The expected development in the flood plain can reasonably be assumed to occur along the floodway fringe. This development can also be assumed to be continuous along that fringe. This assumption is consistent with the assumed complete elimination of conveyance in the floodway fringe when computing water surface profiles for floodways. It is possible that the random filling of the fringe, resulting in localized flow contraction and expansion, may cause greater losses and higher water surface elevations than those computed assuming continuous fill.

Given the idealized study reach, the development of an adequate model is required. The development of the model is first controlled by the physical study reach. Cross sections, reach lengths and Manning's "n" values are selected based on an engineering analysis of the reach. Initial stages of model development usually focus on the input requirements and data entry for the model (computer program) used. Model development is a two stage process. The first stage, collecting the necessary data and assembling it into the proper form for the computer program used, gets most of the attention. The first phase is essential; however, the second phase, model refinement and calibration, is equally important.

Once the data set is developed and running with the computer program used, model development and calibration moves into an analysis process that requires a good understanding of the study reach, the model used, and the underlying theory of the model. While the initial data set was based on the physical characteristics of the study reach and a visualization of the flow's path, this phase of calibration must verify that the computed results are reasonable and consistent with the original assumptions. Additional cross-sectional data and refinements are usually required to develop the model into a reasonable representation of the flow and energy loss process. Also, the computation of historic flood profiles should be used to calibrate the model and provide confidence that it can adequately reproduce observed flood profiles.

The key to a successful floodway computation is a well developed and calibrated model. With adequate care in the model adjustment and calibration phase, the floodway computation should be a relatively straight forward engineering task. The HEC-2 computer program has several options to estimate the limits of the floodway fringe and the resulting water surface profile. If the model has been adequately developed and the study reach reasonably reflects the characteristics of the "ideal," then the options in HEC-2 should quickly provide the limits of encroachment without exceeding a change in water surface elevation limit.

The rush to estimate a floodway can cause the use of an input data set that has not been adequately refined. As soon as the data set seems to work with the computer program, an attempt is made to compute the floodway and complete the job. The computational problems with the existing condition model are compounded with the addition of encroachments. Often, the encroachment process produces erratic results, and sometimes the program cannot even complete the computations.

While problems resulting from incomplete data development are not the subject of this document, it is important to keep in mind that basic data problems may be the source of some of the difficulties encountered in determining floodways. Some guidelines for HEC-2 output review are presented in Training Document No. 26, Computing Water Surface Profiles with HEC-2 on a PC (Bonner, 1987).

4. HEC-2 COMPUTER PROCEDURE

4.1 GENERAL APPROACH

The HEC-2 floodway procedure is based on calculating a natural profile as the first profile in a multiple profile run. Other profiles, in the run, are calculated using various encroachment options, as desired. The six encroachment methods available are described in the HEC-2 Users Manual, Appendix II "Floodway Encroachment Calculations." The input data organization and associated output statements are also presented in Appendix II. An example application of Encroachment Methods 4 and 5 is provided in Example 4, shown in the Users Manual, Appendix I. The input data requirements are defined in Appendix VII. The **ET** record provides the primary input for the encroachment options.

The procedure described herein uses **Encroachment Method 4**, which provides an initial estimate of the floodway encroachments based on a target change in water surface elevation. For encroachment computations, the program stores the computed water surface elevation (CWSEL) and conveyance (Q1) from the first (existing conditions) profile. On the second (or subsequent) profile, the input would specify Method 4 and a "target" change in water surface elevation. The program adds the "target" elevation increase to the first profile water surface elevation and computes the conveyance, which will be greater than it was at the lower elevation. The program then computes the encroachment stations that should reduce the conveyance remaining, at the higher water surface elevation, to the value computed for the first profile water surface elevation. The encroachment computations associated with Method 4 are shown in Appendix A of this document.

The program then modifies the cross section to eliminate the conveyance areas beyond the computed encroachment stations (floodway fringe area). With the section modified, the program computes the water surface elevation for the encroached condition. The resulting water surface elevation may be higher, or lower, than the target increase. The effects of downstream changes and the effect of the redistribution of flow for the current cross section may cause the computed water surface elevation to be different than the target.

Encroachment Methods 5 and 6 are extensions of the Method 4 procedure, and could be used in the same fashion as the following Method 4 example. Both Method 5 and 6 compute the initial encroachment stations using the Method 4 procedure. However, after computing the water surface elevation for the encroached cross section, the results are compared to the initial target. Method 5 compares the change in water surface and Method 6 uses the change in energy elevation. If the computed results do not check with the target increase, the program adjusts the desired conveyance reduction and computed encroachment stations and repeats the process. A maximum of 20 iterations will be used in an attempt to match the target. However, only the current cross section is operated on. Because the computed results at the current cross section are also dependent on the results from the downstream cross section modifications, the program may not be able to match the target increase at the current cross section. Method 5 works best on low velocity streams with gradual changes; while Method 6 works better with high velocity streams.

Recognizing that the initial floodway computations may provide changes in water surface elevation greater, or less, than the "target" increase, initial computer runs are usually made with several "target" values. (For this example, both an 8/10ths and 10/10ths (one) foot targets were used.) The initial computed results are analyzed for increases in water surface elevation and for changes in velocity and other parameters. Also, plotting the computed results on a plan-view map is recommended. From these initial results, new estimates are made and tried. After a few initial runs, the encroachment stations become more defined. Because portions of several computed profiles are used; the final computer runs are usually made with Method 1 defining the specific encroachment stations at each cross section.

4.2 EXAMPLE PROBLEM

The following is an example of the use of encroachment Method 4 in computer program HEC-2 to develop a designated floodway. Data input has been kept simple to illustrate the techniques. Table 4.1 shows a partial input listing. Appendix B contains the HEC-2 output for the problem. A description of the additional encroachment input data and the output are given below. The input and summary printout for a second trial are shown in Appendix C. Both Method 4, with varying targets, and Method 1 encroachments are used in the second trial.

Only a few added input parameters are required to use the encroachment options in HEC-2. The added encroachment data are discussed here; the data listing in Appendix B contains remarks explaining the added data. Reference should be made to the HEC-2 Users Manual, Appendix II for a general description of all the encroachment methods and Appendix VII for the input data description. The Users Manual information is not presented here.

Flow Distribution is recommended when computation water surface profile for floodway determinations. With flow distribution requested, the program prints the lateral distribution of percentage of flow (which equals percentage of conveyance), area, and velocity in the incremental overbank areas. The **1988 version of HEC-2** also provides the hydraulic depth in each increment and provides the distribution in the channel element, if the use of varying channel "n" values causes the channel conveyance to be incrementally computed. The flow distribution information is printed immediately after the normal printout for each cross section.

Table 4.1 Partial HEC-2 Input File Showing Encroachment Data

```
T1    FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2    First Profile is Existing Condition; Method 4 with 0.8 & 1.0 in 2nd & 3rd
T3       COW CREEK NEAR PALO CEDRO
*        * * Remarks Indicate Added Encroachment Input * *
*   INQ (J1.2) specifies field to read on ET and QT
*   Second field of ET = 0 indicating no encroachment
*
J1      0       2       0       0       0       0       0       0     392       0
*
*   J2.10 = 15 requesting Flow Distribution for first profile
*
J2      1       0      -1       0       0       0       0       0       0      15
*
*   Summary tables 110 and 200 are Pre-defined Floodway Data Tables
*
J3    110     200
*
NC    .05     .05    .045      .1      .3       0       0       0       0       0
*
*   1-percent Chance Flood Discharge is defined on all QT fields to be read
*
QT      3   53000   53000   53000       0       0       0       0       0       0
*
*   ET.2 = 0 indicates No Encroachment on first profile (J1.2 = 2)
*   ET.3 = 8.4  "  0.8 foot rise using Method 4 when J1.2 = 3 in second profile
*   ET.4 = 10.4 "  1.0 foot rise using Method 4 when J1.2 = 4 in third profile
*
ET      0       0     8.4    10.4       0       0       0       0       0       0
*
*   No additional ET records required if no change in scale or method desired.
*
X1    .08      21     300     530       0       0       0       0       0       0
GR    410       0     400      40     390     300     380     330     372     350
GR    368     360     368     480     372     490     380     520     390     530
GR    395     700     390     820     380     870     375     890     380     940
GR    390     960     395    1000     390    1100     390    1160     400    1200
GR    410    1500       0       0       0       0       0       0       0       0
X1    .21      18     150     390     750     750     800       0       0       0
GR    410       0     400     130     390     150     380     175     373     200
GR    369     210     369     300     373     320     380     380     390     390
GR    395     600     390     680     385     900     390     980     400    1040
GR    403    1100     395    1400     400    1600       0       0       0       0
X1    .34      16      50     275     650     690     670       0       0       0
GR    410       0     390      50   373.5     100     370     110     370     210
GR  373.5     220     380     250     390     275     395     500     395    1500
GR    390    1720     390    1780     400    1820     403    2200     398    2900
GR    400    3300       0       0       0       0       0       0       0       0
X1    .55      13      20     350    1100    1100    1100       0       0       0
GR    410       0     390      20   374.5      90   371.5     100   371.5     200
GR  374.5     210     380     280     390     350     395     600     395    1200
GR    400    2200     405    2350     400    2900       0       0       0       0
X1    .86      13     120     480    1550    1400    1500       0       0       0
GR    420       0     400     120     376     200     372     210     372     370
GR    376     380     400     480     405     550     400     600     395     800
GR    395    1700     400    2050     410    2400       0       0       0       0
X1    .94      14      75     255     400     400     400       0       2       0
GR    410       0     408      50     383      75     380     110     367     120
GR    367     210     384     255     390     275     392     276     397     300
GR    400     680     398     800     400    1050     408    2000       0       0
X1   1.06      10     190     450     700     600     650       0       0       0
```

* Remarks are added to highlight encroachment related data *

Flow distribution is called by setting the variable **ITRACE equal to 15**, in the tenth field of the **J2** record (**J2.10**). This will provide flow distribution information for every cross section in the profile. The option is usually requested for the first profile of a floodway run because that profile represents the existing condition run. The flow distribution provides helpful information for determining the appropriate location for the encroachment stations. In the example problem, flow distribution is called in the first profile (see Table 4-1 for the input and Appendix B, page B-4 for an example of the output).

Summary tables can be defined using the **J3** record. Two pre-defined floodway tables (**110 and 200**) are available in HEC-2, and the separate summary output program, SUMPO. Table 200 was designed to provide results similar to the format specified in the Guidelines (FEMA, Sep. 1985). The HEC-2 Users Manual, Appendix II provides a variable listing for the two summary tables, and Appendix VII describes the J3 input. In the example problem, both tables are requested.

The primary encroachment input is specified on the **ET** record. All encroachment methods can be specified on the ET; while only Methods 1 and 2 can be specified on the **X3** record. Also, the ET provides the capability to allow a mixture of existing and encroached (floodway) conditions in the same multiple-profile computer run. The ET is read the same as the QT record; that is, the **INQ** variable on the second field of the **J1** record (**J1.2**) defines the field to read on every ET and QT record in the data set. If the QT record is used to define the discharges, the 1-percent chance flood discharge would be input in all QT fields read. Often, every field of the QT is encoded with the appropriate discharge so that any field can be specified by INQ for the encroachment options.

In the example, Method 4 is specified on the ET record with 0.8 foot (8.4) and 1.0 foot (10.4) rise in water surface elevation. (see Table 4.1 Partial Input Listing) The ET field to read, specified on the J1 record (J1.2), is 3 for the first profile and the third field of the ET record is blank. This input specification will produce an existing condition water surface profile for the first profile, which is required for Encroachment Methods 3 through 6. The computed water surface elevation and conveyance from the first profile is used by those methods to compute the encroachment stations. The INQ input on the second and third profiles indicate field 4 and 5, respectively; which will provide the two encroachment profiles.

The starting water surface elevation to be used for the encroachment profiles could be the same elevation as the natural profile if the starting elevation is fixed by a lake, bay, or a channel control. If the study reach is part of a stream in which future downstream encroachments could cause a rise in water surface elevation, then the encroachment profiles should be started at the higher water surface elevation. For this example, the starting elevations are assumed to be equal to the starting elevation for the natural profile plus the one foot increase.

The following listing shows the ending portion of the input data file, which requests the multiple profile computation. Note the starting elevation is set at the 1 foot higher elevation of 393. The **J1.2 (INQ) = 3** to read the third field of the ET record, thus requesting a Method 4 encroachment with an 0.8 foot increase in water surface elevation. Flow distribution was not requested; however, it can be requested for every profile, if desired. The third profile input is similar to the second, except **J1.2 (INQ) = 4** requesting Method 4 with a 1.0 foot rise and **J2.1 (NPROF) = 15** requesting summary printout.

Table 4.2 Partial HEC-2 Input File Showing Multiple Profile Data

```
GR   410      0    400     75    390    190    377    240    373    250
GR   373    400    390    450    395    500    400   1050    407   1600
X1  1.27     14    550    820   1050   1050   1050      0      0      0
GR   410      0    400    280    390    550    380    590    378    610
GR   374    620    374    710    378    720    380    775    390    820
GR   400    920    405   1200    400   1500    408   3000      0      0
EJ
T1  FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2  Encroachment Method 4 with an 0.8 foot increase
T3       COW CREEK NEAR PALO CEDRO
*
*  J1.2 = 3 reads the third field of ET; Method 4 with 0.8 foot increase
*  J1.9 = 393 indicating a 1-foot higher starting water surface elevation
*
J1     0      3      0      0      0      0      0      0    393      0
*
*  No Flow Distribution requested for this profile (J2.10 = 0)
*
J2     2            -1      0      0      0      0      0      0      0
T1  FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2  Encroachment Method 4 with a 1.0 foot increase
T3       COW CREEK NEAR PALO CEDRO
*
*  J1.2 = 4 reads the fourth field of ET; Method 4 with a 1.0 foot rise
*  J1.9 = 393 indicating a 1-foot higher starting water surface elevation
*
J1     0      4      0      0      0      0      0      0    393      0
*
*  J2.1 = 15 requesting the Floodway Summary Tables specified on J3
*
J2    15      0     -1      0      0      0      0      0      0      0
   BLANK
   BLANK
   BLANK
ER
```

* Remarks are added to highlight encroachment related data *

__The output for the Example Problem__ is shown in Appendix B. The added encroachment output is described in Appendix II of the HEC-2 Users Manual. Table 4.3, below, shows an example from the second profile encroachment. The first added statement, numbered **2800**, gives the conveyance data computed by the program and the second, numbered **3470**, provides the computed encroachment stations. Following the encroachment output, the standard output variables are provided for the cross section. The computed solution reflects the cumulative effects of the encroachments.

The order of the output reflects the order of the HEC-2 computations. The **2800** output line shows the conveyance (**Q1**) and the water surface elevation (**WSEL**) for the first, assumed natural (**NAT**), profile. The conveyance under encroached conditions (**ECN Q1**) and the higher water surface elevation (**WSEL**) are also shown on the first line. The **RATIO = 0.0000** indicates that the two conveyance values are equal; that is the encroached cross-section conveyance at the higher water surface elevation equals the natural conveyance at the natural water surface elevation.

The second line shows the conveyance in existing cross section (**NAT Q1**) at the higher water surface elevation (**WSEL**) and the distribution of conveyance in the cross section (**RATIOS LOB, CH, ROB =**). This reflects the intermediate computation step where the water surface elevation is raised the target amount (0.8 foot in the second profile) and the conveyance is recomputed. The increase in conveyance is computed as a ratio and the value is shown as **TARGET** in the **3470** line. One-half of the target amount is removed from the two extremes of the cross section (floodway fringe), if there is sufficient conveyance. The available conveyance is show as **RATIO LOB & ROB**.

If there is not sufficient overbank conveyance, the encroachment station will be set equal to the bank station and the residual TARGET amount will be removed from the other overbank area. The program will not set the encroachment stations inside the left and right bank stations. The computed **ENCROACHMENT STATIONS**, left and right, are shown on the **3470** line along with the Encroachment Method (**TYPE**) and the ratio of conveyance reduction (**TARGET**). After the cross section is adjusted, the water surface elevation is computed and the output reflects the results of the encroachment.

Table 4.3 Sample Encroachment Output from the Second Profile

```
CCHV=     .100 CEHV=     .300
*SECNO  .080
 2800 NAT Q1=  12528.17  WSEL=    392.00  ENC Q1=  12528.17  WSEL=    392.80  RATIO=   .0000
 NAT Q1=  13654.  RATIOS LOB,CH,ROB=   .0028   .7925   .2047  WSEL=    392.80

3265 DIVIDED FLOW

3470 ENCROACHMENT STATIONS=       300.0     916.8  TYPE=      4  TARGET=        .082
      .08    25.00   393.00      .00   392.00   394.35     1.35      .00      .00   390.00
   53000.       0.   45594.    7406.       0.    4630.    1418.       0.       0.   390.00
      .00      .00     9.85     5.22     .000     .045     .050     .000   368.00   300.00
  .001746       0.       0.       0.        0        0        0      .00   500.84   916.84
```

The Summary Printout, starting on page B-16, reflects the choice of variables and tables defined on the J3 record. For this example, the two pre-defined summary tables were selected. **Table 110** is designed to show the computed water surface elevations (**CWSEL**) and the change in elevation (**DIFKWS**) between the first "existing conditions" profile and the following encroached "floodway" profiles. HEC-2 saves the first profile water surface elevations as the "known water surface" (KWS) and computes the difference (DIF) with the following encroached profiles. The **DIFKWS** column makes it easy to see if the change in elevations exceed the maximum criterion for the floodway study. The energy elevation (**EG**) is also provided.

The top width (**TOPWID**) and distribution of flow in the three flow elements (**QLOB, QCH, QROB**) provide a "plan view" of the floodway results. The **PERENC** variable shows the ratio of conveyance reduction and computed encroachment stations (**STENCL** & **STENCR**) are displayed next to the bank stations (**STCHL** & **STCHR**).

Table 200 is designed to present the floodway results in a form similar to that required for a FEMA floodway study report. The **WIDTH, SECTION AREA**, and **MEAN VELOCITY** data are for the entire cross section under floodway conditions. The **WATER SURFACE ELEVATION** data are all rounded to the nearest tenth in a way that ensures that the rounded difference between the elevations is consistent with the rounded elevation values.

Floodway output review tends to focus on the change in water surface elevations because that is the primary criterion for most floodways. However, the entire computer run should be reviewed for reasonableness of results for both the existing conditions profile and encroached profiles. The impact of the encroachment on top widths and velocities should be considered along with change in water surface elevations. Chapter 5 reviews general floodway application problems, and Chapters 6 through 10 present "typical" application problems associated with several categories of study reaches.

The increase in water surface elevation will frequently exceed the "target" used to compute the conveyance reduction and encroachment stations for the section. In the example problem, see Appendix B, the results from the second profile with an 0.8 foot target increase in elevation produced water surface elevation increases greater than 1.0 foot in portions of the reach. That is why several target increase values are generally used with initial floodway computations.

Seldom are the encroachment results from a single profile acceptable for the entire reach; however, portions of different profiles usually are acceptable. The second, and succeeding, trials are usually based on combinations of the multiple-profile results, defined in a single profile. As the floodway model is refined, **it is important to remember that the change in water surface elevation at a cross section reflects the current and**

previous cross-section modifications. If one section has an increase in water surface elevation well above the target change, it usually means that the previous cross-section modification is contributing to the current section's large increase. The amount of encroachment at the previous cross section, as well as the current, should be reduced.

In the Second Trial, shown in Appendix C, the first profile is still the "existing condition." The second profile uses **Method 4** to compute the encroachment stations, but the target change in elevation was varied in an attempt to keep the computed change in water surface elevation within one foot. The third profile uses **Method 1** to specify the encroachment stations. The stations were selected based on the computed stations provided in the First Trial and the resulting change in water surface elevation. The **ET input** for the two approaches are illustrated in the partial data listing, shown below.

Table 4.4 Partial Input Listing for Trial 2

```
T1   FLOODWAY DETERMINATION EXAMPLE - Second Trial -
T2   First Profile Existing Condition; Second Method 4, & Third Method 1
T3        COW CREEK NEAR PALO CEDRO
J1       0       2       0       0       0       0       0       0     392       0
J2       1       0      -1       0       0       0       0       0       0      15
J3     110     200
NC     .05     .05    .045      .1      .3       0       0       0       0       0
QT       3   53000   53000   53000       0       0       0       0       0       0
*
*     ET.3 requests Method 4 with 0.9 foot rise
*     ET.4 indicates Method 1 with Encroachment stations in fields 5 and 6
*          Selected stations (300 & 920) were based on the First Trial results.
*
ET       0       0     9.4     5.1     300     920       0       0       0
X1     .08      21     300     530       0       0       0       0       0       0
GR     410       0     400      40     390     300     380     330     372     350
GR     368     360     368     480     372     490     380     520     390     530
GR     395     700     390     820     380     870     375     890     380     940
GR     390     960     395    1000     390    1100     390    1160     400    1200
GR     410    1500       0       0       0       0       0       0       0       0
*
*     ET.3 changes the Method 4 target to 0.8 foot rise
*     ET.4 defines the Encroachment stations for the next cross section.
*          Method 1 only applies to the following cross section; therefore, an
*          ET is required before each corss section when using Method 1.
*                                        STENCL  STENCR
ET                     8.4     5.1     150     820
X1     .21      18     150     390     750     750     800       0       0       0
GR     410       0     400     130     390     150     380     175     373     200
GR     369     210     369     300     373     320     380     380     390     390
GR     395     600     390     680     385     900     390     980     400    1040
GR     403    1100     395    1400     400    1600       0       0       0       0
*                                        STENCL  STENCR
ET                     6.4     5.1      50     350
*
X1     .34      16      50     275     650     690     670       0       0       0
GR     410       0     390      50   373.5     100     370     110     370     210
GR   373.5     220     380     250     390     275     395     500     395    1500
GR     390    1720     390    1780     400    1820     403    2200     398    2900
GR     400    3300       0       0       0       0       0       0       0       0
*
*     ET.3 is blank, indicating no change from previous value of 6.4.  All the
*          Methods, except 1, apply continuously, unles changed by a new entry.
*                     BLANK              STENCL  STENCR
ET                             5.1      20     500
```

As the computed floodway is refined, plot the floodway on a plan-view work map. While the analysis describe here focuses on the HEC-2 output and the change in water surface elevation, it is important to remember that the floodway must be consistent with local development plans and provide a reasonable hydraulic transition through the study reach. Sometimes a computer solution, that provides a floodway with most computed water surface elevations at or near the maximum, may be unreasonable when transferred to the actual study reach.

5. FLOODWAY APPLICATION PROBLEMS

With the ideal floodway application described, some of the common sources of computational and application problems are presented. In some cases a study reach may have aspects of several problems, and in other cases, the problems may not fit any of the descriptions that follow. The following paragraphs describe some of the problems encountered in the application of the floodway concepts.

5.1 MODEL ASSUMPTION PROBLEMS

The computational process used in computing floodways is outlined in Section 3.3, and the key computational assumptions are listed in Section 3.4. These assumptions are the basis of many application problems and deserve a separate review and comment.

a. **Steady flow** assumption is not a significant problem in floodway determination. For most naturally occurring floods on major streams, flow changes slowly enough with time that steady flow is a fair assumption. Even when it is not, the assumption would seldom cause any computational problems.

b. **Gradually varied flow** may not be a valid assumption in the vicinity of manmade structures such as bridges and channel control structures. The estimation of energy losses in more rapidly changing flow becomes more uncertain; therefore, the computed water surface elevation is more uncertain too. Under these conditions, the estimated energy loss may be too high or low, or it is possible that the computational process will not be able to determine a water surface elevation. Without a solution based on computed energy losses, the HEC-2 program generally assumes critical depth. For most flood plain studies, the critical depth solution is not valid. Also, a critical depth solution at a cross section will not provide a basis for computing a floodway encroachment based on a change of water surface elevation at that section.

c. **One-dimensional flow** may also not be a valid assumption. The location and definition of cross sections requires a considerable degree of understanding of the flow pattern to properly define the input data. Two major problems that violate the assumption of one-dimensional flow are: (1) multiple water surface elevations and (2) flow in multiple directions.

1. **Multiple water surface elevations** within one cross section usually result from multiple flow paths. When the flow in each path is physically separated from the other paths, the distribution of flow in each path is a function of the conveyance (or energy loss) through the length of that path. Because the one-dimensional model distributes flow in each cross section based on the conveyance in that section, the flow distribution in the model is free to shift

from section to section in the computational process. The traditional solution to the problem is to divide the model into the separate flow paths and compute a profile for each one (see Chow, 1959, Sec.11-9).

2. **Flow in multiple directions** cannot easily be modeled with a single cross section perpendicular to the flow. In cases where the flow is gradually expanding, contracting, or bending, the section can be defined that will reasonably meet the requirement, but it does take special care. When flow takes a separate path, like a levee overflow or a side diversion, then the flow lost from the main channel must be separately estimated and subtracted from the main channel flow. The HEC-2 program has a Split Flow Option to compute lateral flow losses and the resulting profile in the main channel (HEC, Sep. 1982).

d. Channels of small slope are common in natural streams. A slope less than 1 in 10 means that the pressure-correction factor is close to 1 and, therefore, not required. Also, the depth of flow is the same whether measured vertically or perpendicularly to the channel bottom (Chow, 1959). For most valley streams where floodway computations are performed, a 1 in 10 slope would be considered steep. Channel slopes are usually less than 1 in 100.

c. Rigid boundaries mean that the channel shape and alignment are constant for the period of analysis. The concern is not with the long term changing boundaries, like with meandering rivers. More important to the floodway analysis is the local scour and deposition that can occur in a stream during a flood event. The problem is generally more pronounced at major contractions, like bridge crossings, because there is a general increase in velocity and, therefore, potential for increased scour. General velocity guidelines can be found in design criteria for stable channels (USACE, 1970).

Modeling scour and deposition would require considerably more data, engineering expertise, and a movable bed computer model. A National Research Council study concluded that rigid-boundary models should be utilized for flood insurance studies until deficiencies in the application in movable bed models are removed (NRC, 1983). The principal deficiencies were inadequate understanding and formulation in the mechanics of the various processes, and a general lack of available input data.

5.2 FLOODWAY COMPUTATION PROBLEMS

There are numerous computational problems that result from limited and/or incorrectly formulated model data. The basic data for floodway water surface profile calculations are:

a. Discharge (peak discharge for the 1% chance flood)
b. Flow regime (usually subcritical)
c. Starting water surface elevation

d. Cross sections (defining flow boundaries)
e. Reach lengths (between given cross sections)
f. Loss coefficients (Manning's "n" and contraction/ expansion coefficients, if used)

With the limited budget available for the typical flood plain study, there is a natural tendency to collect a limited amount of data, especially cross sections. Once the data are collected and formatted as computer program input, there is little chance to obtain additional data. Because the programs will execute with the limited data, the computations can proceed. Also, there is the question of how much data is required to compute the water surface profiles with sufficient accuracy.

In an HEC study, 98 HEC-2 data sets were used to evaluate water surface profile accuracy (HEC, 1986). The data, from previous flood plain studies, were edited to produce consistent data sets without bridges or non-surveyed cross sections. Comparisons made of profiles computed using several commonly used friction loss approximation techniques "...show significant differences, more than a foot, in reaches of many streams. A significant number of the original data sets underestimate the profiles as compared to those calculated with more accurate integration of the energy loss-distance function made possible by using closer-spaced cross sections" (Burnham & Davis, 1986, pg.5). The study used a cross-sectional spacing of 500 feet, by interpolating cross sections from the survey data, to calculate the "true" water surface profile. The assumption was that the survey data adequately defined the physical study reach, but the added sections were needed to improve the energy loss computations.

These study results indicate that limited cross-sectional data, in the extreme, may cause computational difficulties that would make computing water surface profiles for flood flows and developing floodways very difficult and probably inaccurate. In a less-severe, data-deficient case, the profiles may be computed but with questionable accuracy. Potential errors of one foot or more could be expected. With sufficient cross-sectional data and careful, efficient use of that data, a reasonable model could be developed. The refinement of the model, through a calibration process, then becomes the next focus.

Calibration is a time consuming task. Historic flood flows and flood level data are required. The model may need several cycles of adjustment and execution before the results reasonably reproduce available historic information. Without this process, the computed profiles have considerable uncertainty. The previously cited study on profile accuracy found that the Manning's "n" values may be the most significant data in the profile computation process. "Significant effort should be devoted to determining appropriate Manning's coefficients" (Burnham & Davis, 1986, pg.22). The adjustment of this model parameter is a major part of the calibration process.

The processes of model development and calibration may be overlooked in the haste to develop the flood plain and floodway data; however, the computational problems that may result from incomplete model development could cause greater delays. If basic computational problems, like those results based on assumptions of critical depth, are not eliminated, the floodway computation will be more difficult and time consuming. A training document for HEC-2 application provides some suggestions for model calibration and output analysis (Bonner, 1987).

Once a model has been developed and calibrated, the floodway computation process can begin. If the model is working well, the floodway computation, based on the change in water surface elevation criterion, should not be a major problem. However, the computed floodway may not be reasonable from an application point of view.

The typical computation of cross section encroachment, based on an equal reduction of conveyance in the overbanks, does not consider the total floodway as a two- or three-dimensional entity. The computation results may yield top widths that increase and decrease from section to section (undulating top widths). The computed encroachment may indicate a floodway that runs contrary to the natural flood plain meander. Additionally, the computations do not reflect the local development plans for the flood plain. Defining the floodway is not simply a computation problem. The computed results require adjustments and additional engineering refinement to define the floodway. The added refinement, and the more judgmental nature of the refinement, is a floodway determination problem.

In a study of the origin, use, and rationale of the one foot criterion, a small number of floodway studies were reviewed. Even though the maximum change in water surface elevation was one foot, the average computed change was 0.7 foot (Goddard, 1978). Also, increases at many points were found to be less than 0.4 foot. This would indicate that the floodway computations did not necessarily define the minimum necessary floodway. Conversations with FEMA staff indicate that there are cases where subsequent computations have shown that additional encroachments could be made, into what has been defined as floodway in previous studies, without exceeding the one-foot increase in water surface elevation. This has encouraged the emphasis on determining the <u>maximum encroachment</u> when computing the floodway.

Computing a floodway which defines the maximum limits of encroachment may require several iterations and considerable engineering judgment. Bridges and other obstructions to flow complicate the problem. Because a bridge may act as a local control, the change in water surface elevation upstream from the bridge may be quite different than the downstream change. It may take several cross sections before the profile approaches the maximum allowable change in water surface elevation. With more bridges, or other obstructions, the problem is compounded.

The refinement process could continue for many cycles of adjusting the stations that define the limits of encroachment and then computing the water surface profile to determine the impact of the estimated floodway. In those states with additional criteria for floodway determination, the process will also require evaluating the results based on the added requirements. The problem becomes increasingly complex when local development plans and the more subjective desire for a natural floodway are added. In summary:

a. The computation of floodways in a complicated problem.

b. Proper data development and parameter calibration are essential to the floodway computation.

c. Many computational problems can result from inadequate data and improperly used models.

d. The model computations may not consider all the aspects that should be considered.

e. Beyond the initial floodway computations, there are numerous, more subjective refinements that could be made.

5.3 PROBLEM CLASSIFICATION

Grouping floodway problems into general categories does not provide a complete classification system; however, there are certain fundamental problems that cause difficulty when applying the current procedure. These application problems were generally defined and placed into the following categories:

1. Low gradient streams - usually with low velocity, long duration floods over a wide area.

2. Flood overflow situations - including overflow at drainage divides and on leveed streams.

3. Alluvial streams - with movable boundaries.

4. High velocity streams - flowing at supercritical and subcritical velocities.

5. Developed flood plains - with development in the potential floodway zone.

While it is extremely difficult to define general solutions to unique and variable application problems, major common problems have been defined. The floodway concept applicability and general solution approach are provided in the following chapters.

6. LOW GRADIENT STREAMS

Low gradient conditions frequently occur at the lower end of the stream's course, as it approaches its outflow point. Low gradient streams are commonly located in wide flood plains in coastal areas and may be subject to tidal influence. Numerous examples exist in the Gulf States. Not only is the stream slope low, around 1 in 1000, but the stream overbank area is usually wide and flat. These characteristics yield low velocity flood flows over wide areas. The computed floodways are often wide too. Based on this description, the low gradient stream appears similar to the ideal application described in Section 2.5. Computing the floodway limits should not be a major problem, given a well developed and calibrated model.

6.1 APPLICABILITY OF FLOODWAY CONCEPT

The general nature of the wide flood plain is an extensive inundated area, often with shallow flood depths and low velocities. The flow conveyance for much of the overbank area may be very low, and, therefore, the conveyance lost due to development in that area may be relatively small. A significant modeling problem for these flood plains is the determination of the conveyance and storage zones in the flood plain. (A modeling approach is described in Section 6.2.) Once an area is defined as a storage zone, without conveyance, it is automatically a part of the flood plain fringe. The steady flow, water surface profile computation does not consider the storage zone in determining the flow conveyance and water surface profile. Therefore, development in the storage zone will not increase the computed water surface elevation.

Loss of overbank storage can influence the peak discharge and travel time of a flood wave. While most floodway computations do not evaluate the storage effect, it should be recognized. The downstream peak flow will tend to be higher and arrive sooner with a filled overbank area, as assumed for floodway determination (DeVries, 1980). It deserves mention here because the wide flood plain is more likely to have a significant amount of overbank storage and, if significant storage area is loss due to flood plain encroachment, the potential effect on the flood wave should be evaluated.

The floodway computation is based on completely eliminating conveyance from the fringe of the conveyance portion of the cross sections. If development were to occur in that fashion, then the computations should reasonably reflect the cumulative effect of eliminating the flow carrying capacity of that fringe area. Therefore, the current floodway concept is applicable to the low gradient, wide flood plain areas. The computation assumptions and procedures can define an area (floodway) required to pass the base flood without increasing the water surface elevation by more than a designated amount. However, an alternative procedure could meet the requirements and also be more consistent with the likely development in the flood plain.

While the computation of a floodway may reflect the impact of the complete elimination of the flow carrying capacity along the flood plain fringe, the question of whether the computation reflects flood plain development remains open to debate. What portion of the lands, adjacent to the river, is required to discharge the base flood? The difficulty occurs when the computed floodway is compared with the flood plain development that has occurred, or is expected to occur. The computed floodway may meet the change in water surface elevation criterion and not produce any hazardous conditions. However, the floodway may also be very wide and include areas that are locally seen as the best areas for development. Adopting the computed floodway may be the major problem, not the computation of the floodway.

6.2 FLOODWAY COMPUTATION PROBLEMS

A typical problem in model development for wide flood plains is the definition of the cross sections. Because the flood plain is usually very flat, the limits of the cross sections are not apparent. The question is often: "How far out should the cross section extend?" The limits of effective flow are not obvious and the wider the cross section, the more data required.

Aerial photos of historic floods can help define the limits of flooding. Also, photos can indicate where water appears to be effectively moving and the direction of flow. This information can be used in locating sections perpendicular to flow. Flow may be 2-dimensional in a wide overbank. If photos are not available, then field interviews and other sources of historic information are often helpful when defining the cross section limits.

Once the field data has been obtained and the input data developed, the calibration process begins. One modeling problem with the wide flood plain is the definition of the limits of effective flow in the cross section. The nature of the modeling problem is outlined below:

1. In the one-dimensional model, flow in the cross section is distributed proportionally to conveyance. The section elements can be considered as the channel and the two overbank areas. (Some computer programs may consider more elements, but they usually consider at least the three primary elements.)

2. When the discharge is high enough to flood the overbank areas, the overbank flow area (and conveyance) is usually very large, when compared to the channel area.

3. The distribution of flow, based on conveyance, places a large proportion of the total discharge into the overbank area. The channel flow proportion is reduced, accordingly.

4. The computed flow in the channel, can be far less than the flow was at bank full discharge. Also, the computed velocity in the channel will be less than it was for lower discharges, when all the flow was in the channel.

A solution to the flow distribution problem is to reduce the overbank conveyance, which will cause more of the total discharge to be in the channel. The problem is that this adjustment is often handled in an arbitrary fashion. In some cases, a fixed lateral distance is used when the cross-sectional data are obtained. In other cases, the average velocities in the incremental areas of the flood plain fringe are used to limit the conveyance. A typical criterion is to eliminate areas with a velocity less than one foot per second. The eliminated area is considered a storage zone, not used for conveyance calculation.

A more reasonable approach is to reduce the overbank conveyance until the flow in the channel produces channel velocities near the value obtained when the entire flow was in the channel. There is a general increase in channel velocity with an increase in flow and depth. So a channel velocity near, or slightly greater than, the bank-full channel velocity would be expected when the flow is higher and moving into the overbank area. The difficulty with this approach is that it may take several trials to get the distribution "balanced" for the design flood.

There are two methods available to modify the overbank conveyance: either block (or eliminate by some method) the ineffective overbank fringe or raise the Manning's "n" values in the ineffective area (See Figure 6.1). When the overbank area is eliminated, the true values for flood plain width, flooded area, or volumes are not reflected in the computed results. If those items are important, then high "n" values should be used to reduce conveyance in the ineffective areas. Then high "n" values allow the water to be located in the ineffective area, but the computed conveyance will be negligible because it is inversely proportional to the Manning's "n" value.

The method used to redistribute, or limit, the overbank conveyance should not affect the floodway computation directly. The usual approach, in computing the floodway, is to eliminate the overbank conveyance, compute a new profile, and evaluate the change in water surface elevations. If the fringe of the flood plain has been adjusted to eliminate or reduce its conveyance, the adjusted area is not effective and will not impact on the computed floodway profile. That is, if it was not considered as effective conveyance in the existing condition profile, then eliminating it in the floodway profile will not affect that profile either.

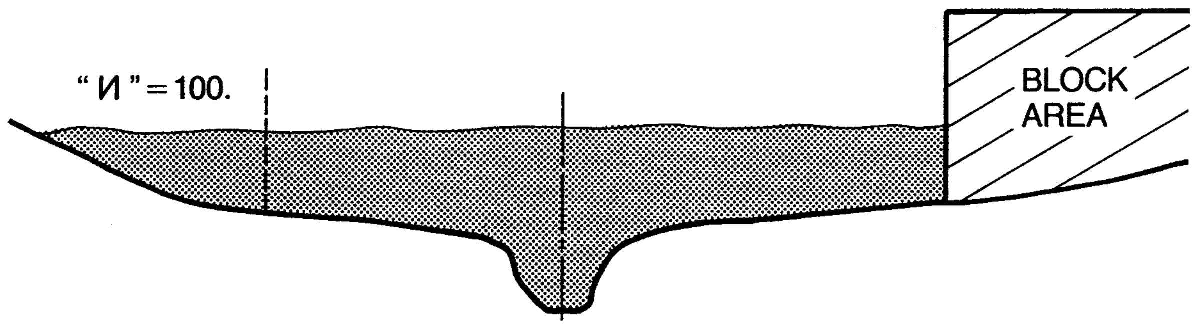


Figure 6.1 Modifying Cross-section Flow Area

The use of high "n" values to define ineffective area does not limit the floodway computations. Also, by using high "n" values, an estimate of valley storage can be obtained from the water surface profile computations. A storage-discharge function can be developed by computing profiles for a range of discharge values and relating the discharge to the storage under the computed profile. The storage-discharge function can be computed for the existing flood plain and for a proposed floodway. By routing the base flood hydrograph through the study reach for the two conditions, an estimate of the impact of lost storage can be made.

If there are no other physical problems in the study reach, the primary computation problems in computing floodways in low-gradient, wide flood plain streams pertains to definition of the cross sections and the limits of effective flow. In a few extreme cases, even the direction of flow may be uncertain.

6.3 FLOODWAY APPLICATION PROBLEMS

Floodways computed in these applications tend to be wide, and may not fit well with the community's view of the development potential of the study area. Part of the problem comes from the assumptions in the computation process. The floodway is determined by eliminating the floodway fringe from the outside limits, toward the channel bank, until the change in water surface elevation reaches the target. Conveyance is removed equally from both sides of the flood plain. Nothing in the existing procedure considers potential, or attractiveness, of any particular portion of the cross section for development.

With the equal conveyance, continuous elimination of the floodway fringe there may be instances where portions of the excluded floodway fringe may have more conveyance than portions remaining in the floodway. Because conveyance is a function of flow area, the deeper portions of the overbank can generally be considered to have greater conveyance than the shallow areas. An examination of the cross-sectional areas in the fringe would clearly show where the greater depths may be excluded.

An added practical problem comes from the requirement that the development occur above the base flood level. Because there are additional costs involved in raising the first floor elevation above the natural ground level, the portions of the cross section with the most shallow flooding would be the least costly to develop. With this in mind, it is obvious that a floodway that allows filling and development in some deeper fringe areas, while preserving some more shallow flood plain area, would not be very attractive to the local community planning to develop the area.

The floodway concept in wide flood plains in currently under review. FEMA is considering alternative procedures for some communities.

7. FLOOD OVERFLOW SITUATIONS

There are a variety of situations that could be characterized as flood overflow situations. For this discussion, the topic is limited to situations where a portion of the flood flow leaves the general path of the channel and proceeds down a separate course. The location can be a natural drainage divide or a manmade structure. The portion leaving the main channel may or may not rejoin the the channel flow downstream.

The case where the natural stream course is higher than its overbank area (perched streams) is also considered. For perched streams, the overflow may occur in several places, and it may be more difficult to predict the locations of overflow without profile computations. Leveed channels are also discussed in this section, primarily as a computation problem. FEMA has guidelines for levee evaluation (FEMA, Sep. 1985).

7.1 APPLICABILITY OF FLOODWAY CONCEPT

The application of floodways to study reaches with areas of flood overflow is complex. The degree of applicability depends on the nature of the overflow, the impact of flood plain encroachments on the magnitude of the overflow, and the interaction with the local community on the acceptability of the potential alternatives. There is no single answer to the problem.

In an overflow situation, a primary question is whether the overflow proceeds into a storage area or into a secondary flow path. Development in storage areas will not directly affect the water surface elevations along the conveyance channel. Therefore, the conveyance-based floodway computation would not apply to the storage area.

Overflow into a storage area reduces the peak discharge in the primary channel. One example would be a levee break. The reduced discharge would result in a lower water surface profile downstream from the overflow. For example, a simulation of the flood caused by Tropical Storm Agnes in the Susquehanna River basin estimated that the downstream peak discharge would have been 10 percent greater if the levee had not been overtopped at Wilkes-Barre, Pennsylvania (Feldman, 1973).

Development in areas that provide flow-carrying capacity would directly affect the water surface profile. In overflow situations, there may be considerable difficulty defining the flow paths, the distribution of flow through the various paths, and the impact of development along the flow paths. However, the floodway concept is applicable to those situations. Development along the flow paths can have a significant impact on the base flood elevation.

The following discussion describes floodway concept application for three overflow situations: (1) at a drainage divide, (2) on a perched stream, and (3) on leveed streams.

7.1.1 OVERFLOW AT A DRAINAGE DIVIDE. For a reach containing a natural drainage divide, when flow depths exceed a controlling elevation, a portion of the flood flow leaves the study reach and proceeds in a different direction. In some cases, the drainage divide can be a manmade flood bypass system, e.g., the Yolo Bypass of the Sacramento River. The manmade system is not the concern here because for such systems the flood overflow is known and usually managed. The natural overflow situation may not be known at the start of a study, especially if recent flood events have not exceeded the controlling elevation and a careful review of the historic flooding situation has not been performed.

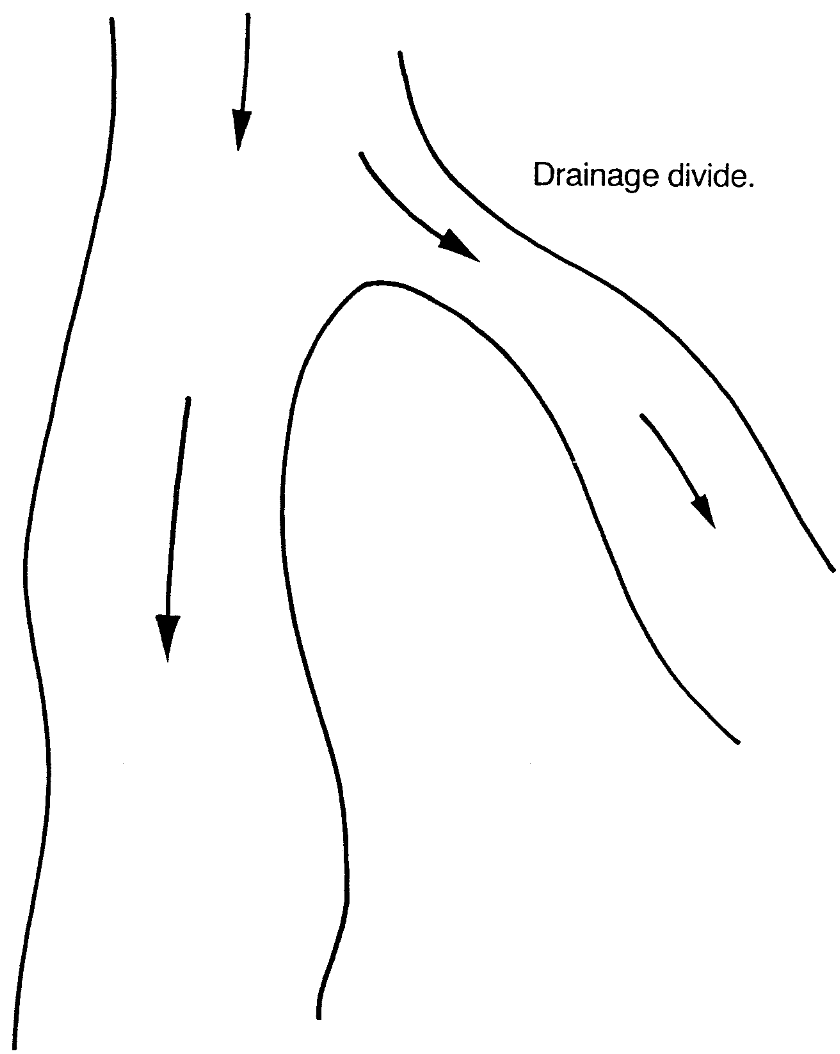


Figure 7.1 Overflow at a Drainage Divide

The next concern is whether the flood overflow occurs at the elevation of the base flood, or if it occurs with a simulated floodway. In either case, the floodway computation will increase the amount of overflow by raising the water surface elevation. However, if the overflow does not occur at the base flood level, but would at the increased elevation resulting from a floodway, there is the added issue of increased flooding in an area that would not have been flooded without the floodway. The floodway computations should be based on the controlling elevation for overflow as the target elevation. This approach still increases the risk of flooding in the overflow area for flood events higher than the base flood.

The current floodway procedures can be applied to this flood plain situation. Two major problems need to be solved in order to apply the procedure: (1) estimate the lost flood flow and (2) provide for the passage of the lost flow. Methods for estimating the lost flow are discussed in Section 7.2. Providing for the passage of the overflow must be negotiated with the appropriate local authority. The overflow passage must be considered as a portion of the floodway to preserve its conveyance. The traditional floodway computations would then be performed for the study reach with the remaining portion of the base flood as the design discharge.

The area flooded during overflow conditions should be delineated as a portion of the flood plain. If it is a storage area, the area flooded depends on the volume of overflow and the elevation-storage characteristics of the flooded area. In effect, it is a reservoir storage problem. The computational tools are available to solve the problem; however, they do require additional data that probably would not be developed for the typical floodway study. The computations require the entire flood hydrograph for the base flood, while usually only the peak discharge is estimated. Development in the flooded area would generally reduce the flood storage available, but it would not be significant in most situations. Therefore, the floodway concept would not apply to the overflow storage area. The primary concern should be to define the area flooded.

7.1.2 OVERFLOW ON A PERCHED STREAM. Perched streams result from the development of natural levees from sediment deposits. The stream may overflow at any number of locations, and the overflow water generally moves laterally away from the channel. The definition of the flood plain and the floodway is very difficult. The applicability of the floodway procedure may depend on the degree of overflow, which may not be known until initial modeling has been performed.

If the channel overflows at discharges well below the base flood level, conventional floodway computations might be applied easily. The key is whether the one-dimensional assumption is appropriate. The question is: Whether a single water surface elevation can be assumed across the entire flood plain. If the answer is yes, then the floodway concept should be applicable. Under this condition, the flood flow should be contained so that the flow is following the general path of the channel. (Flow lines are generally parallel.) However, if the answer is no, the problem is more difficult. The degree of difficulty depends on the number of locations where the water overflows the channel and where the water goes once it leaves the main channel.

Once the water overflows the channel and flows in a different direction than the main channel, the one-dimensional solution is no longer applicable. Some additional computations are required to estimate the quantity of overflow and the area flooded. If the overflow locations are limited in number and definable in a model, the computational problem is similar to the drainage divide problem. Once the overflow estimates have been made, defining the potential floodway may be easier. The key to floodway applicability is whether the overflow area is a conveyance or storage area. Section 7.2 presents the major computation problems and suggested solution procedures.

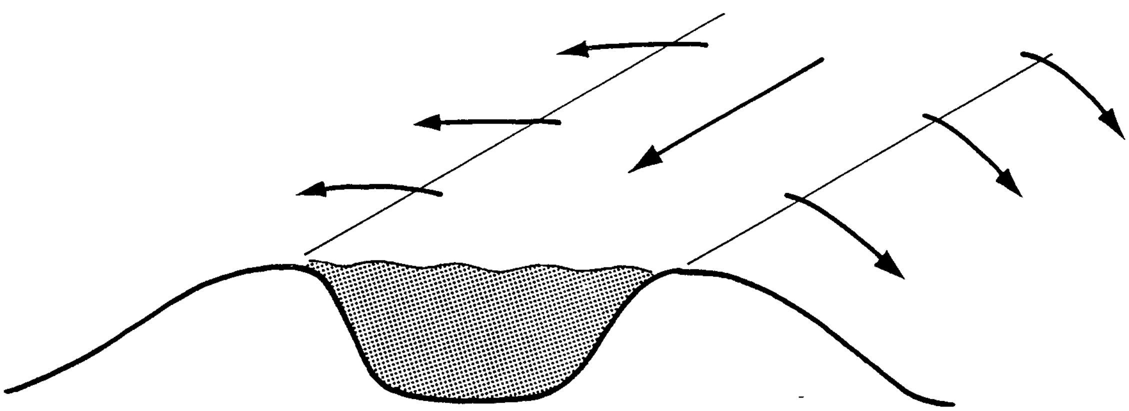


Figure 7.2 Overflow on a Perched Stream

7.1.3 OVERFLOW ON A LEVEED STREAM. The application of the floodway procedures for a leveed stream is similar to that for a perched stream. The primary difference is the evaluation of the levee to verify that it provides a 1-percent chance flood level of protection. The Guidelines require certification of ownership, maintenance, and sufficient freeboard to ensure that the levee is likely to perform at the required level of protection in the future (FEMA, Sep. 1985). If the levee does not meet the required criteria, the area protected by the levee will be evaluated as if the levee did not exist. For the area outside of the levee protection, a base flood profile, at the point of levee overtopping, would also be computed, and the higher of the two profiles would be used to define base flood elevations.

Floodways are delineated at the landside toe of credited levees (FEMA, Sep. 1985). There is no floodway computation. Development could occur in the protected area without any impact on the base flood conveyance.

For leveed streams that do not meet certification standards, the base flood elevations are computed without the levees controlling the flow. Therefore, a floodway could be computed based on this same assumption. The flood plain and floodway would be modeled by the one-dimensional water surface profile computation, assuming water can freely move between channel and overbank.

7.2 FLOOD OVERFLOW COMPUTATION

There are three basic questions to answer when computing water surface profiles for streams with potential overflow problems.

a. Is there an overflow problem?

Compute water surface profiles assuming there is no overflow problem. Review the computed results and determine whether the computed water surface elevations exceed any controlling elevations in the study reach. If they do, there is an overflow problem.

b. How much water is "lost" due to overflow?

Model the overflow situation to determine how much water is lost and how much remains in the study reach. The modeling approach depends on the nature of the overflow. Three cases are considered:

CASE 1 - Overflow is confined and limited.
CASE 2 - Overflow is general and fills the overbank.
CASE 3 - Overflow is general and not confined.

c. What happens to the overflow water?

It may be necessary to model what happens to the overflow water in order to define the area flooded. Three cases are presented:

CASE 1 - Water is contained in a storage area.
CASE 2 - Water is conveyed in a separate flow path.
CASE 3 - Water is conveyed in an uncontrolled path.

The general procedures used to estimate flow lost due to overflow, and the resulting inundation from the overflow, are discussed below. The application of those procedures to the types of overflow encountered is presented in Section 7.3.

7.2.1 IS THERE AN OVERFLOW PROBLEM? Using the one-dimensional model approach, the water surface profiles would be computed without considering any overflow. In many cases, the fact that there is an overflow is unknown at this point in the study. The computed profile would indicate water surface elevations exceeding the channel boundary, and the general location and nature of the overflow would be known. The computed profile can be used to estimate the amount of water lost by using the computed water surface elevation to estimate the potential energy necessary to move the water over the overflow location.

The overflow computations can be done externally to the water surface profile computation, and then the profile can be recomputed with the remaining discharge (adjusted based on the estimated overflow) in the study reach. The HEC-2 computer program has an option for making these iterative computations automatically. The Split Flow Option is described and demonstrated in the HEC Training Document No. 18 (Montalvo, 1982). The general computation procedure is described in Section 7.2.2.

An alternative approach would be to use a two-dimensional model to evaluate the transfer of flow through the drainage divide. Generally, the two-dimensional modeling would increase the study effort two to ten times that required for a one-dimensional model. However, a more complete solution can be obtained, and if the full hydrograph is available, the entire flood hydrograph could be routed through the model. This more complete computational approach would give both volumes and discharge values for the two flow paths.

7.2.2 ESTIMATING WATER "LOST" DUE TO OVERFLOW. If the computed water surface profiles indicate that a portion of the flow would pass into a separate flow path, then the problem is to estimate the flow loss. While there are two-dimensional models available, the following procedures are based on the assumption that the typical flood plain analysis would be limited to the one-dimensional model.

Case 1 - Overflow is confined and limited in area, for example, a drainage divide. In this situation there would be a limited number of cross sections where the computed water surface elevations exceed the cross-sectional boundary. At the computed elevations, some of the flow would leave the study reach and cross over the boundary.

The computed water surface elevations can be used to estimate the potential for flow to move out of the study reach. The physical nature of the overflow geometry would provide an indication of the appropriate model to estimate the amount of flow that would leave the reach. Typical models would be the weir equation applied to the overflow section or channel conveyance based on the overflow path. The determination of what is controlling the overflow amount and the appropriate model is an engineering decision.

The HEC-2 Split Flow Option can be applied to a variety of overflow situations. The overflow discharge can be modeled with a weir flow equation, a rating curve, or by using normal depth. A weir model would apply to lateral overflow along a levee, and normal depth would be used when the overflow was controlled by the conveyance of the overflow path. The rating curve would usually be used to model a side diversion on the channel. The overflow is computed based on the computed water surface elevation, and the channel discharge is reduced to reflect the overflow. A new profile is computed with the modified flows, and the computed elevations provide new estimates of overflow. The program iterates until the assumed and computed flows agree within the error criterion.

The steady flow solution obtained by this procedure assumes that there is sufficient flow volume to maintain the discharges. As a result, this procedure is most applicable to a very broad-based flood with a peak discharge maintained over a considerable period of time. If the flood is fairly rapid in its rise and fall, the lost flow may be far less than the quantity estimated by the steady flow solution. Therefore, this approach is most applicable to large basins with large volumes of runoff.

To get an estimate of the volume of lost flow, the entire flood hydrograph is required. In the typical floodway study, a flood hydrograph is not developed; instead, an estimated peak discharge value and steady flow computations are used. However, if a flood hydrograph is available, the Split Flow Option can be used to develop a rating curve for the overflow locations, and those curves can then be used to estimate the time-distributed overflow by treating them as diversions in a flood routing simulation. The HEC-1 Flood Hydrograph Package can perform the simulation (HEC, 1985). The computation would provide estimates of the volume of lost flow and the remaining discharge in the main channel. The National Weather Service's hydrodynamic model DWOPER can provide dynamic wave routing and solve the overtopping problem, based on a weir flow model (Fread, 1982).

Case 2 - Overflow is general and fills the overbank, for example, low level levees. If two conditions are met, the problem is easy to solve. The first condition is that the overflow is general and fills the overbank area. The second is that the overbank flow follows the same general path as the channel.

If the levee elevation is exceeded at a level well below the 1-percent chance flood discharge and the overbank area is fully occupied by the overflow, the modeling problem is not any different than the ideal floodway condition. In this situation, the levees merely block a portion of the cross-sectional area. With the water surface profile well above the levee elevation, the water should be able to move in and out of the overbank area, depending on the profile elevation. If the overbank area is sufficient to control and maintain the flow in the flood plain, there would not be any additional computational problems.

There is a range of flows, from bank full to the level where the overbank areas are flowing full, where the one-dimensional solution is not appropriate. The flow exceeds the channel capacity, but the overbank conveyance is greater than required to carry the overflow. The one-dimensional solution allocates the cross-sectional flow proportionally to the conveyance and cannot balance the solution with the flow in the overbanks. The HEC-2 program provides a message indicating that it cannot obtain a balanced solution. The correct solution requires an estimate of the overflow and the computation of the water surface profile based on the remaining channel flow. If the overflow is confined to a flow path, a water surface profile can be separately computed based on the estimated overflow discharge.

A second situation where the one-dimensional solution fails is when the flood flow in the overbanks does not move parallel with the main channel. The perched stream is an example. The problem becomes a more difficult two-dimensional problem.

Case 3 - Overflow is general and not confined to an identifiable flow path. The overflow area can be extensive, and the overflow water may move into different flow paths. Examples include perched streams with overflow going into broad flow areas, and leveed streams with flow passing over several portions of the levee.

As long as the overflow location can be considered stable, the HEC-2 Split Flow approach can be used to estimate the overflow. The weir model for overflow calculations would be preferred. It is difficult to apply the normal depth, conveyance-based split flow computations to a broad overflow area.

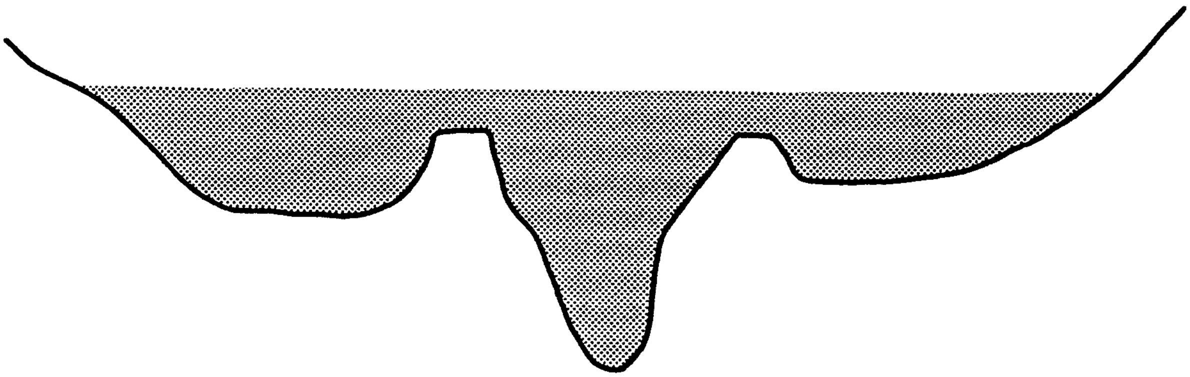


Figure 7.3a Overflow Fills the Flood Plain
(One common water surface elevation applies)

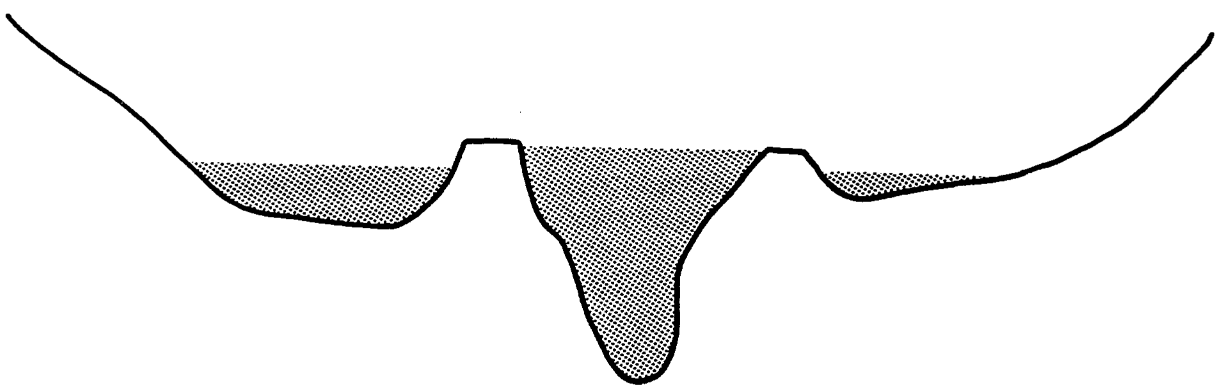


Figure 7.3b Overflow Partially Fills the Overbanks
(Multiple water surface elevations exist)

The Split Flow option is not suitable for divided flow situations with complex multiple flow paths. If the flow is confined in multiple flow paths, the individual paths can be analyzed separately following the procedure described in Section 11-9 of "Open-Channel Hydraulics" (Chow, 1959) or in Appendix III of HEC Training Document No. 18 (Montalvo, 1982). If the flow is not confined, the solution would require a two-dimensional model, which is beyond the scope of the typical floodway analysis.

The overflow computations may require numerous overflow reaches and several trials to obtain a solution. The profile calculations are from downstream to upstream, but the overflow loss would occur upstream first. Additional cross sections and overflow profile definition may be required to obtain an adequate solution. The "Split Flow Reach Length Consideration" Section of Training Document No. 18 (Montalvo, 1982) provides information on the effect of reach lengths on the overflow computation.

7.2.3 ESTIMATING THE AREA FLOODED BY OVERFLOW. Once the amount of lost flow has been estimated, it may be necessary to determine the area flooded by the overflow. The cases considered are: (1) water is contained in storage; (2) water is conveyed in a separate flow path, and (3) water is conveyed in an uncontrolled flow path.

Case 1 - Water is contained in a storage area adjacent to the overflow location. The physical boundary of the overflow area prevents the water from moving through a separate flow path. The flooded area depends on the volume of overflow water; therefore, the entire hydrograph is required to estimate the volume. The flood hydrograph could be routed with an unsteady flow program, like DWOPER (Fread, 1982), or the steady flow solution from Split Flow calculations can be used, as follows.

First, a rating curve for the overflow location(s) is developed. The rating curve can be developed based on split flow computations for a range of channel discharges. Then a routing model (e.g., HEC-1 Flood Hydrograph Package) is used to route the flood hydrograph through the study reach. The overflow is modeled as a diversion. The overflow rating curve defines the diversion relationship. The computed diversion hydrograph represents the overflow, and the remaining hydrograph represents the downstream channel flow.

If the overflow moves into a low-lying area, the flooded area would be defined by the volume of overflow and the storage capacity of the area. The problem is similar to a reservoir storage problem where the low-lying area is the reservoir without an outflow. How high the water will rise can be determined by the storage-elevation curve for the area. This flooded area should be treated as part of the flood plain. If development is to occur in this area, the limits could be estimated using the one-foot change in elevation criterion based on the reservoir storage. However, defining an overflow area "floodway" based on storage is not recommended. Flood easement or land use control, consistent with paragraph 60.3 of the regulations (FEMA, 1986), should be used to maintain the overflow area.

Case 2 - Water is conveyed in a separate flow path. If the overflow moves into a separate flow path, a separate analysis should be performed to delineate the flooded area. If there is sufficient flow, a separate water surface profile computation could be performed for the overflow. A separate data set would be required to define the overflow reach. The overflow discharge, computed from the primary study reach, would be the basis of profile computations.

The magnitude of the overflow discharge may be different for the with and without floodway conditions on the primary study reach. Presumably, the floodway condition would raise the computed water surface elevations, and the higher elevations would cause more water to move across overflow location(s). Computing overflow for the two conditions may severely complicate the problem. The floodway computation is dependent on the flow, and the flow is dependent on the resulting water surface elevation.

Case 3 - Water is conveyed in an uncontrolled path, for example, overflow from a perched stream. If the flow is moving across a broad area, a one-dimensional model usually cannot adequately model the flow. The one-dimensional solution requires cross sections based on the flow directions and the expected lines that define equal water surface and energy elevations. If the flow is in several directions, the problem requires a horizontal two-dimensional model. Approximate flow depths can be estimated by treating the overflow area as a flow plane. The flow plane approach would be consistent with the "Shallow Flooding" procedures in the Guidelines (FEMA, Sep. 1985).

7.3 FLOODWAY APPLICATION PROBLEMS

The floodway concept is applicable to the three situations described in Section 7.1. Application problems with the current floodway procedures are reviewed for each situation in the following sections.

7.3.1 DRAINAGE DIVIDE OVERFLOW. The overflow can be estimated using the procedures described in Section 7.2.2 for Case 1. Once the distribution of flows has been determined, the floodway computation can be performed based on the remaining flow in the study reach. The floodway computations could proceed as simply as the ideal flood plain application, except for the impact of the higher floodway water surface elevations on the overflow discharge.

The higher floodway water surface elevations would provide an increased energy level for the overflow. With the higher head, the overflow quantity would increase. Therefore, the distribution of flow with and without the floodway would be different. The difference depends on the elevation change and the overflow characteristics of the drainage divide.

The floodway computations can be performed on the study reach using the current procedures and the remaining estimated discharge, after overflow discharge is subtracted from the base flood discharge. A flood bypass zone will have to be defined to provide for the passage of the overflow discharge into the bypass area. Proper development control will be required, and the continued management of the bypass area will have to be assured in order to support the study reach floodway.

The bypass area can be delineated based on the procedures described in Section 7.2.3. If the area receiving the overflow acts as a storage area (Case 1), the limits of flooding under base flood conditions should be defined as part of the flood plain delineation.

If the overflow is conveyed through the bypass area (Case 2), then the conveyance area should be defined as a separate flood plain that is treated as an integral part of the study reach floodway. A separate floodway could be computed for the bypass area. However, the problem becomes more complex when you consider the interaction of the overflow calculations with the different water surface elevations with and without floodways in the primary study reach and the overflow area.

There is a potential trade-off between the study reach and the bypass. When the study reach floodway is computed, the higher water surface elevation may increase the flood overflow and increase the area flooded in the bypass. In effect, the flooded land in the bypass is traded for the developed land along the study reach. This trade off can be evaluated, and a potential compromise solution negotiated with the local community.

7.3.2 LOW-LEVEL LEVEES. The application of the floodway results should not be a major problem for this situation, if the overflow is contained in the overbank area (Case 2). The computation of the floodway should be nearly ideal if the natural terrain controls the overbank flow in a course parallel with the main channel flow. The one-dimensional solution, assuming that a common water surface elevation applies across the entire cross section, would apply.

If the overbank does not control the flow along the channel flow path (Case 3), the problem becomes two-dimensional. For cases where the overflow is limited to a definable path or area, the procedures and concepts presented in the sections on drainage divides apply. If the overflow is not limited to a definable region, then the situation is similar to sheet flow, and the floodway concept does not apply. Delineation of sheet flow areas is usually limited to defining the overflow region and the approximate depth.

7.3.3 HIGH-LEVEL LEVEES. While the floodway delineation for certified levees is clearly defined by the levees, the floodway application is not used for those levees that do not meet the criteria for 1-percent chance flood protection. By assuming the levees do not control the flow, the base flood elevation can be computed with the levees allowing the water to freely move in and out of the overbank areas. This approach is suggested in the Guidelines. For low level levees, which are exceeded by flood flows well below the base flood, the assumption of a one-dimensional model may be appropriate. However, for levees that provide nearly 1-percent chance flood protection, the assumptions of a one-dimensional model may not be appropriate.

Modeling the high levee stream requires assumptions about the levee's performance during a major flood. Will the levee be overtopped at a known location, and will the levee withstand the overflow? If the answers are yes, the procedures for drainage divides could be used. However, what if the overflow causes the levee to fail? Then the overflow assumptions would not correctly model the outflow, and the flooded area would be underestimated. Also, there is the possibility that the levee might be overtopped in another location, and a different area would be flooded.

While there are procedures for estimating the overflow, the uncertainty of the actual performance of the levees make the modeling process uncertain. The Guideline approach does not require the determination of where the levee will fail or what its performance will be under overflow conditions. The computed base flood elevations reflect a general failure of the levee system. The floodway would provide the necessary conveyance under this condition.

7.4 SUGGESTED FLOODWAY PROCEDURES

The floodway concept is applicable to most overflow situations. If there is water loss due to overflow, then the estimated amount of overflow and the area flooded by the overflow should be determined. Floodway computations would be performed based on the flow remaining in the study reach, after subtracting the overflow loss.

Procedures are available to compute overflow in drainage divide and levee situations. Section 7.2.2 describes the computations using the HEC-2 Split Flow Option. Additional information can be obtained in the HEC Training Document No. 18 (Montalvo, 1982). Using these procedures, the remaining flow in the study reach can be estimated and a floodway can be computed using the current procedures. The flood plain and floodway delineation must include provisions for the passage of the overflow and allowance for its storage or passage. Section 7.2.3 provides procedures for estimating the area inundated by the overflow.

The sheet flow condition, which might occur with overflow on a perched stream, may be an exception to the above procedures. The Guidelines, Appendix A2, provide general procedures for sheet runoff computations. For conditions where sheet flow will be one foot or less, only the area flooded is defined; detailed studies are not required. For those conditions where depths are expected to exceed one foot, but be less than three feet, estimating the depth of the sheet flow using normal depth approximations is suggested. The use of traditional floodway computations would not be appropriate for the sheet flow area. The requirements of paragraph 60.3 of the Regulations (FEMA, 1986) would apply.

The Split Flow procedure would not be applicable to flow separation into two or more paths, like divided flow around an island. If the flow is confined in multiple flow paths, the individual paths can be analyzed separately following the procedure described in Section 11-9 of "Open-Channel Hydraulics" (Chow, 1959) or in Appendix III of HEC Training Document No. 18 (Montalvo, 1982). If the flow is not confined, the solution would require a two-dimensional model which is beyond the scope of the typical floodway analysis.

Floodway computations are applicable for streams with levees that do not meet the Guideline requirements for certification. The one-dimensional model solution is consistent with the assumption that levees do not control the flow, and water is free to move in and out of the overbank areas. If the study reach, ignoring the levees, controls the flow then the current floodway computation procedures should be applicable. For those locations where the levees provide nearly a 1-percent chance flood protection, the floodway delineation may appear unreasonable. In those situations, the floodway would be some distance outside the levees.

8. ALLUVIAL STREAMS

Alluvial streams are authors of their own geometry and are continually adjusting their dimensions (slope, depth and width) through the processes of aggradation and degradation (deposition and scour) in response to the present flow conditions and sediment transport characteristics. Therefore, potential analysis problems, dealing with flood plains and floodways are related to the traditional assumptions that channel boundaries are rigid (immobile) and static (will not change over time).

Some alluvial channels may be relatively inactive, and standard floodway procedures are adequate. However, many alluvial channels, especially in the arid Southwest, demonstrate a great deal of mobility and readjustment during and after flood events. For these types of channels, alternative floodway computation procedures may be necessary to accurately delineate floodway boundaries and flow characteristics.

Once disturbed, an alluvial stream begins an automatic and unrelenting process that culminates in its reaching a new equilibrium with nature. The new equilibrium characteristics may or may not be similar to the stream's original characteristics or channel features. Failure to recognize important sediment transport characteristics of an alluvial stream can lead to a situation in which computed water surface elevations are not accurate, estimated channel velocities are exceeded and manmade channel modifications require expensive periodic repair.

The purpose of this chapter is to discuss those situations where standard floodway procedures are inadequate and to present alternative techniques and procedures that may provide more reliable information for flood plain management.

8.1 APPLICABILITY OF FLOODWAY CONCEPT

Rigid boundary assumptions, upon which most flood control and flood insurance studies are currently based, do not acknowledge the potential for river systems to move both laterally and vertically. Failure to address this natural characteristic of alluvial streams in the design and construction of flood control projects, river crossings or other structures or channel modifications located within the flood plain can lead to their damage, destruction or premature obsolescence.

Additionally, if sediment transport related problems are not addressed properly during the evaluation of potentially active alluvial channels, long-term upstream and/or downstream changes may occur as the channel attempts to readjust itself to new equilibrium conditions. Determination of design fault and damage liability issues is not straightforward, but is certainly becoming a more common occurrence. If significant morphological changes occur in the channel over time, results from past flood insurance and floodway delineation studies may become invalid, necessitating expensive new studies. According to Linder (1976), "In the past, too many problems associated with flood control, drainage, navigation and other types of water resources projects have been handled by modifying the river channel involved without giving equal thought to the sediment being transported by the water flowing in those channels." Misunderstanding the role of water in transporting enormous quantities of sediment has been more a matter of neglect than unawareness.

Flood insurance studies and the floodway analyses associated with these studies are not necessarily concerned with designing or building projects as part of the study. They are primarily concerned with the delineation of floodways and flood hazard areas within the flood plain so flood hazard maps can be developed and flood insurance rates can be estimated.

The floodway concept is based on being able to reasonably define the distribution of flood flow in a flood plain. Those areas that can be developed with a minimal impact are defined and become the basis for land use zones and development control. The applicability of the floodway approach must be based on the present ability to reasonably assess the conveyance for various alluvial stream types. Therefore, prior to performing a detailed floodway analyses, one must try to answer the following questions:

a. Does the study reach reasonably resemble and possess the characteristics of an "ideal floodway" as described, and will it satisfy the assumptions listed?

b. Will water surface profiles and velocities computed by traditional procedures be valid? If not, can one estimate the approximate amount of departure away from the true water surface elevation or velocity that may be contributed to alluvial channel characteristics?

c. Is the study reach "reasonably" stable? That is, if a floodway is developed based on the present channel characteristics, will it remain essentially the same into the planning future? How can engineers quickly determine whether a stream may be active and/or present sediment transport related problems for evaluating the floodway now or in the future?

d. What are possible alternative procedures for computing floodway characteristics in active alluvial streams?

The following sections present some preliminary guidance pertaining to these questions. As the suggested procedures are tried and applied, they will undoubtedly require further improvement to cover the variety of field conditions found across the country. The procedures should, therefore, be considered as preliminary at this time.

8.2 SITUATIONS NOT APPLICABLE FOR FLOODWAY COMPUTATION

There are several situations where the floodway concept is difficult to apply. The following conditions often fail to sufficiently meet the criteria for ideal floodway application. Generally, the assumption of a rigid boundary is not applicable, which makes it difficult to apply the traditional floodway procedures. Many of these situations can be defined as erosion-prone areas, which are included under paragraph 60.5 of the Regulations (FEMA, Oct. 1986).

Alluvial Fans - An obvious flood prone area that frequently fails to meet the criteria of an ideal floodway is the alluvial fan. Alluvial fans develop below the mouth of a canyon by the outwash of sediment and debris from the canyon draining an upstream watershed. Flooding on an alluvial fan is part of the natural process that forms the fan. Usually, the channel is poorly defined on an alluvial fan. Even if there appears to be a channel, the flows during a flood event may leave the channel and form new ones (avulsions).

The procedures provided in Appendix 5 of the Guidelines (FEMA,Sep.1985) are based on statistical analysis, rather than the floodway procedures described. There is no "floodway." The FEMA methodology is based primarily on the following assumptions that obviously fail to meet standard floodway criteria.

a. On the upper portion of the alluvial fan, the flow is confined to a single channel. Critical flow is assumed which defines the flood velocity and depths.

b. On the lower region of the fan, the flow splits into multiple channels. Normal flow conditions are assumed to exist.

Therefore, given a peak discharge for a flood, the flow depth, channel width and flow velocity are calculated. Risk of flooding is distributed spatially across the fan according to a probability relationship that is related to the radial distance from the apex of the fan. Velocity and depth are considered in defining zones within the flood hazard area. FEMA, the Corps of Engineers and others are presently working to improve methods for evaluating flood hazards on alluvial fans. The Arid West Committee of the Association of State Floodplain Managers is also working to develop more appropriate methods and criteria for delineating and regulating development on alluvial fans.

Braided Streams - Braided streams commonly fail to meet ideal floodway criteria. Braided streams are typically the least stable form of alluvial channels. They are relatively wide with poorly defined, unstable banks. They typically possess multiple channels and multiple flow paths within those channels. Channels are very active and frequently migrate over the flood plain. Braided channels usually result from:

a. the stream being supplied with more sediment than it can presently carry, resulting in deposition of its excess load,

b. steep valley slopes which produce a wide shallow channel where bars and islands readily form.

Either, or both, of these factors can lead to braiding. Any floodway developed for a braided stream must be compared to the future size and position that is likely for the channel.

Lateral Migration, Bank Instability and Erosional Hazards - Lateral migration of river channels, river bank instability and erosional hazards associated with geomorphologically active river channels are not considered by traditional FEMA flood plain or floodway analysis procedures. The analysis is based on the assumption of a rigid boundary. However, the banks of a laterally migrating channel may shift dramatically due to bank erosion and bank failure. The lateral shift could cause the channel to relocate into the fringe area of the previously defined floodway.

This type of flood related problem often occurs in the arid Southwest where ephemeral sand bed channels quickly fill with flood waters from thunderstorms. Prior to a storm, completed FEMA floodway maps may include the main channel and only a narrow band of land adjacent to the channel. During a storm, an active alluvial channel and banks may move tens of yards laterally, effectively moving the channel out of its previously located FEMA floodway. This type of movement quickly invalidates FEMA flood plain maps and makes regulation of floodway requirements extremely difficult for local flood plain management agencies.

Methods for determining the likelihood of lateral channel migration are needed. This situation truly violates the assumptions that the channel remains rigid and static during a storm event. Officials with the Arizona Department of Transportation and the Pima County Transportation and Flood Control District estimate that traditional FEMA flood plain maps can become invalid every two years due to lateral movement of their most active alluvial channels (Pearthree, 1987). Both the floodway and the base flood elevations are subject to change.

In-Channel Development - In-channel resources harvesting and development, such as sand and gravel mining, may also cause significant changes in the locations of channel banks, bed elevation, bed slope and channel roughness. Depending upon the extent of in-channel sand and gravel mining activities, channel adjustments upstream and downstream from the mining location may occur for many years as the channel seeks to re-establish its equilibrium. The same kinds of effects can occur following the placement or construction of new channel crossings. Because the long range impacts of these developments are difficult to predict, management of the floodway becomes a difficult task for the local community.

Extreme examples are easily identified. Streams that fit into the middle of the stability classification are more difficult to evaluate. How stable are they, and should static boundary or rigid boundary solutions be applied? In the absence of more detailed studies, historical information and general stream classification methods should be used to make the determination. If the study reach reasonably fits the ideal application, if it does not greatly violate the assumptions presented, and if the history and present condition of the study reach indicate a quasi-stable channel, then current procedures can be applied with reasonable confidence.

8.3 FLOODWAY CALCULATION PROBLEMS

In alluvial channels one should consider the effects of channel characteristics that change with the flow and during the passage of an event. The interaction between the flow of the water-sediment mixture and the sand bed creates different bed configurations that change the resistance to flow and rate of sediment transport. The gross measures of channel flow, such as the flow depth, river stage, bed elevation and flow velocity, change with different bed configurations. In the extreme case, the change in bed configuration from plane to dunes or antidunes can cause resistance to flow to change by a factor of 3 (Simons, Li,& Assoc, 1982, pg.6.16). For a given discharge and channel width, a 3-fold increase in Manning's "n" results in a doubling of the flow depth.

The interaction between the flow and the bed material and the interdependence among the variables make the analysis of flow in alluvial sand bed streams extremely complex.

If a rigid boundary model is not applicable because the channel form changes significantly during a flood event, or because the channel form and location is expected to change significantly over the planning horizon, then why not use a movable bed model? An investigation report, "An Evaluation of Flood-Level Prediction Using Alluvial/River Models" (NRC, 1983), describes a study to evaluate six mobile-bed computer models. A study objective was to determine "whether flood-zoning studies should make use of flood-stage prediction models that incorporate river-bed mobility and degradation/aggradation, instead of utilizing fixed bed models,...." A study conclusion was that the modeling process will continue to be more reliable, but until improvements are made to the cited model deficiencies, "..rigid-boundary models should be utilized for flood-insurance studies,...." The report also recommends examining the sensitivity of rigid-boundary model solutions to "..uncertainties and variations in channel roughness, channel geometry, and channel slope."

Of the three factors mentioned above, the channel roughness is the most easily evaluated and likely to have the most significant impact on the computed water surface profile. The Hydrologic Engineering Center recently completed an investigation to determine the accuracy of computed water surface profiles using HEC-2 for various ranges of channel roughness and geometry changes (HEC, 1986).

A recent report, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" (Arcement & Schneider, 1984), should be helpful in estimating Manning's roughness coefficients ("n" values) for flood plain studies. A step-by-step procedure is provided for estimating the "n" value for channels and flood plains. A section of the report provides procedures for estimating "n" values in sand bed channels considering bed form.

Additional methods for computing the effects of bed configurations (roughness elements) in alluvial channels are presented by Vanoni (1977), Graf (1971) and Simons, Li & Associates (1982). Methods presented in these references can be used to estimate the effective flow resistance associated with different grain-sized bed materials and different bed configurations. The dimension of dramatic bed configurations such as dunes and antidunes may be on the same order as the depth of flow itself, thus violating the computational assumptions presented in Section 3.4.

The accumulation of debris in an alluvial channel can also cause dramatic changes in water surface profiles and local channel velocity. Effects of debris accumulation are not explicitly addressed by traditional computational methods.

In some cases, an estimate of the debris accumulation can be made. The impact on the computed water surface elevation can be evaluated by modifying the cross-sectional definition to reflect the debris blocked area and then computing the water surface profile. The problem of predicting debris accumulation makes it difficult to place any reliability on the computations beyond an assessment of the sensitivity of the profile computation.

If one could reliably estimate the potential impact or amount of influence that changing bed forms or debris may have on the computed water surface elevation and/or channel velocity, then a more reliable floodway could be determined.

8.4 CLASSIFICATION OF ALLUVIAL CHANNELS

As a first step toward determining whether an alluvial stream can be analyzed with traditional floodway methods, one should classify the channel according to its location and its potential for fluvial activity.

All rivers can be separated into two major groups depending on their freedom to adjust their channel boundaries due to flow conditions. Bedrock or non-alluvial channels are confined within, or between, rock outcrops so that the material forming their bed and banks determines the channel morphology. These are geologically controlled channels. Alluvial channels are free to adjust their shape and gradient in response to hydraulic changes. The materials they transport are similar to those materials comprising the bed and banks of the alluvial channel. Alluvial channels are of primary interest because of their dynamic behavior in response to spatial and temporal changes of natural and man-induced processes.

Schumm (1977) presents a generalized definition of an idealized fluvial drainage basin and river system as the linkage of three physiographic zones (see Figure 8.1). In this description, Zone 1 is the drainage basin, upland watershed or sediment source area. Erosion and sediment production are generally the dominant sediment transport processes occurring in Zone 1. Zone 2 is the transfer zone or zone through which Zone 1 sediments are transported. The processes of aggradation and degradation are both active throughout Zone 2. Zone 3 is the sediment sink or region of deposition.

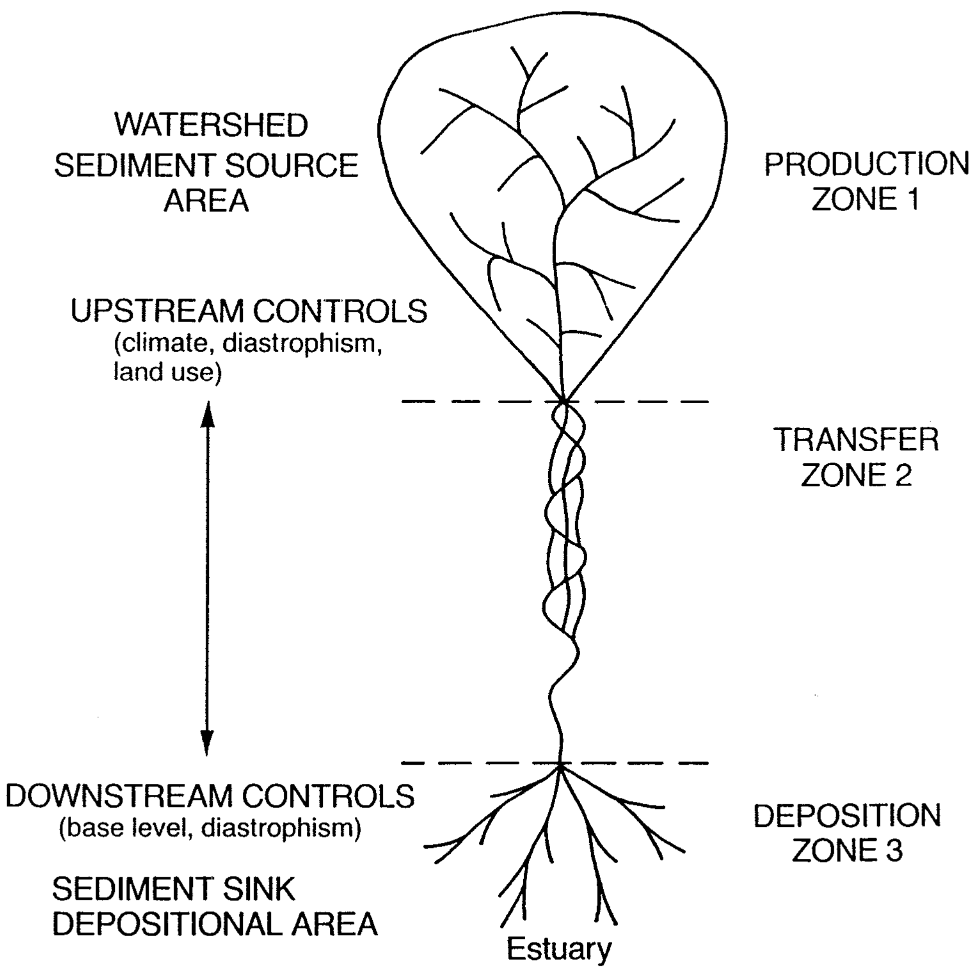


Figure 8.1 Idealized Fluvial System (from Schumm, 1977)

According to Schumm's (1977) definition, Zone 3 is concerned primarily with estuaries and the coastal region since they are considered to be the ultimate deposition zone. Consequently, in this analysis of inland waterways for the purposes of delineating the floodway, Zone 3 (the estuary and coastal zone) generally is not considered for a floodway. Zone 3 would usually be defined as a FEMA Zone V, which comes under paragraph 60.3(e) of the Regulations (FEMA, Oct.1986). The fluvial system of interest here is the interaction of the watershed and alluvial channel network.

Niell (1978) proposes a refinement of Schumm's definition of the idealized fluvial system (see Figure 8.2). All three Zones described by Schumm are still present. However, regions are further classified according to the dominant local sediment transport processes governing the morphology of the alluvial channel, such as erosion, transportation, deposition and ultimate deposition (such as occurs in deltas, lakes, reservoirs, and estuaries). Also shown in Figure 8.2 are sketches of the relative amounts of sediment of different sizes (gravels, sands, silts and clays) typically in motion within the different regional zones in the fluvial system.

Even if the description of the fluvial system with its alluvial channels is limited to these simplified definitions, the system is still highly complex, involving the interaction of many natural processes. These natural physical processes govern the response of the fluvial system to various inputs and/or disturbances. Two primary inputs are climatic factors (hydrospheric forces) and man's activities (attempts to improve or modify the natural characteristics of the system).

The most important climatic factor affecting erosion and sediment transport is precipitation in the form of rain or snow that leads to runoff and increased stage and discharge in alluvial channels. Man's activities include water resources development, watershed conversion, resources acquisition (energy, sand and gravel extraction, etc.), development and maintenance of transportation systems, and development and maintenance of flood control projects.

The response of the fluvial systems to these inputs and/or disturbances is governed by the relevant physical processes operating in the system as shown in Figure 8.2 and the proximity of the channel to these inputs and disturbances. For example, in alluvial channels within the system, the physical processes describing sediment transport capacity balanced with the availability of sediment materials establish whether or not a channel will aggrade or degrade in response to precipitation generated water and sediment runoff.

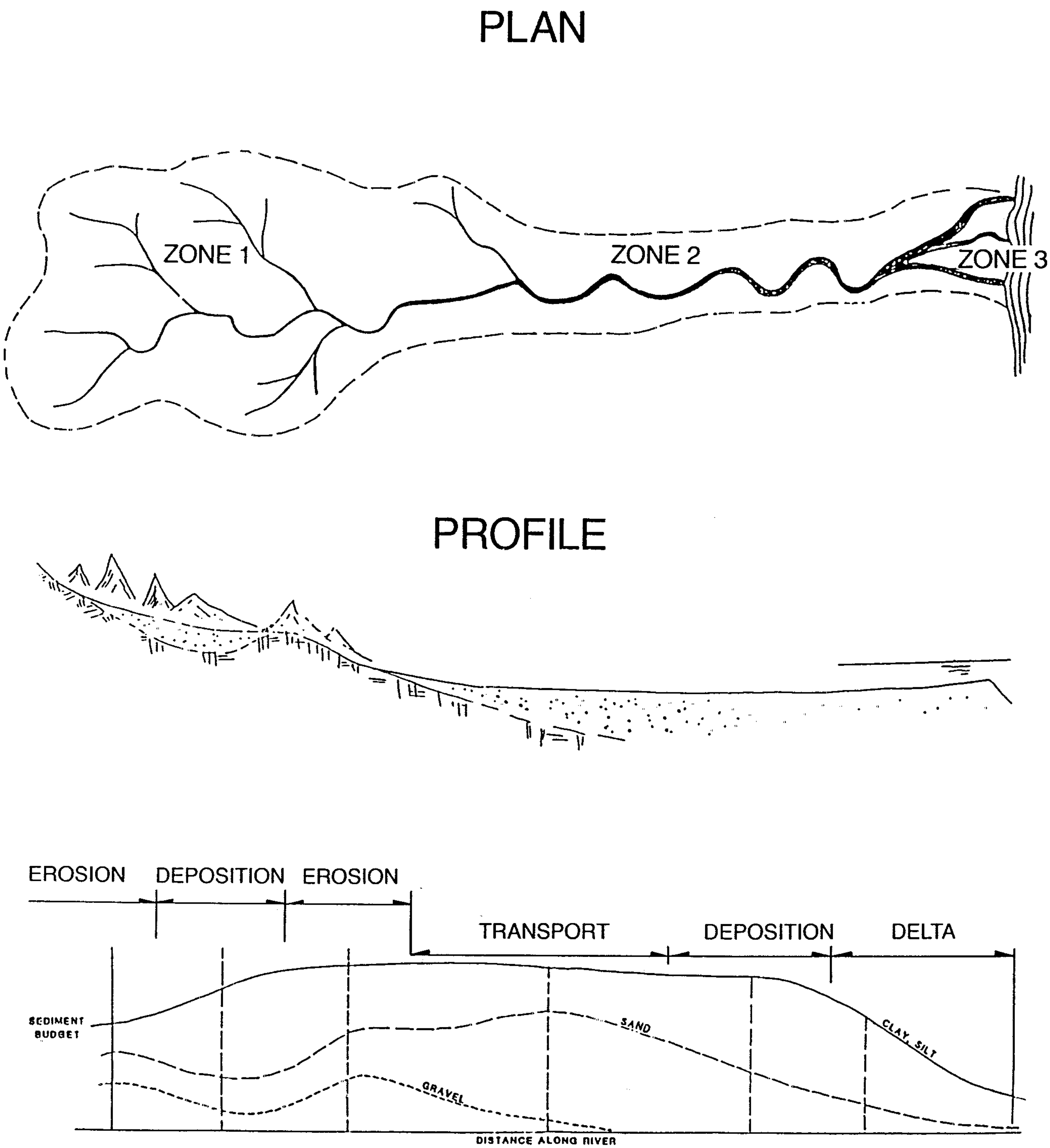


Figure 8.2 Classification of Alluvial Channels (from Niell,1981)

Another useful classification scheme is presented by Simons, Li & Associates (1982). It is especially useful for floodway studies because it defines the stream by channel type such as straight, meandering, braided or some combination of these. In their scheme, the channel progresses from high stability to low stability; from a low channel-width-to-depth ratio to a high ratio; and from a low bed-load-to-total-load ratio to a high ratio. The higher the number that best fits the general channel description for the study reach, the less stable the study reach is.

1. Straight channel with a straight thalweg. (The thalweg is the lowest point in the channel.) The channel is relatively stable.

2. Straight channel with a sinuous thalweg. The channel is relatively stable, but there may be shifts in the thalweg and location of bar formations.

3. Meandering channel. If the channel has a uniform width with small point bars, the channel is stable but neck cutoffs may occur. If the channel is wider at the bends and has large point bars, the potential cutoffs and meander shifts make it a relatively unstable channel.

4. Meandering channel with braided transitions, large point bars and frequent cutoffs. Generally, the channel is unstable with a sinuous thalweg.

5. Braided channel. This is an unstable channel with multiple thalwegs and numerous bars and islands. Islands with more permanent vegetation would indicate a more stable condition than without vegetation.

Lane (1955) developed a simple qualitative relationship that provides a quick and simple tool for estimating whether changes in discharge, sediment load, average sediment size or channel slope will affect the sediment transport capacity of a channel and, therefore, its stability. Simons, Li & Associates (1982) give several examples of how to effectively use Lane's relationship to perform qualitative assessments.

The primary factors affecting channel morphology are water discharge and sediment load. An increase or decrease in discharge (either mean annual or mean annual flood) changes the channel geometry and slope. Channel width and depth are directly related to discharge while channel slope is inversely related. A change in the bed load changes channel dimensions, gradient, width-to-depth ratio and sinuosity (Simons, Li, & Asoc, 1982). The classification system described above does not use discharge because it mainly influences the size of the channel, the type of transported material, and the proportion of bed load.

8.5 CHANNEL STABILITY ASSESSMENT

Changes in channel geometry with time are particularly significant during periods when alluvial channels are subjected to comparatively high flows. The converse situation exists during low flow periods. Erosive forces during high flow periods may have a capacity approximately 100 times greater than those forces acting during periods of intermediate and low flow. It has been estimated that approximately 90 percent of all alluvial river changes occur during that 5 to 10 percent of the time when large flows occur. As mentioned in Section 8.2, the assumptions of static and rigid boundaries may be inappropriate during high flows.

Therefore, assessment of the stability of an alluvial channel for flood flows may indicate whether traditional floodway computations are acceptable. Procedures by Ingram (1986), Simons, Li & Associates (1982), Vanoni (1977), Graf (1971) and Northwest Hydraulic Consultants (1984) can be used to quickly determine the relative stability of an alluvial channel given the general channel dimension, slope, bed material size and range of discharges. Procedures to estimate channel stability based on Shield's diagram for incipient motion, critical bed shear stress and/or critical channel velocity are presented in Chow (1959), Vanoni (1977), Simons, Li & Associates (1982) and are found in "Hydraulic Design of Flood Control Channels" by the U.S. Army Corps of Engineers (1970). These procedures can be applied to evaluate whether channel velocities with and without a floodway may lead to scour and an unstable channel condition.

Effects of channel encroachments on channel velocities and potential increase in scour can also be assessed. Many easy-to-use utility computer packages are available from the "CORPS" library of computer codes to aid engineers in determining the stability of a channel for different flow conditions. The U.S. Army Corps of Engineers at the Waterways Experiment Station provides information and assistance with the "CORPS" computer programs that estimate channel stability and hydraulic characteristics.

8.6 FLOODWAY APPLICATION PROBLEMS

The following is a list of floodway application problems that may occur if traditional floodway procedures are applied to active alluvial channels.

a. Lateral migration and channel shifts toward the floodway limit may threaten the fringe development. Severe lateral shifts may affect areas formerly outside of the floodway zone.

b. Aggradation within a floodway channel will raise the base flood level leading to an increased flooding hazard with time.

c. Braided and laterally migrating streams may require that the floodway width be wider than computed by traditional methods in order to cover the areas of potential flooding (provide channel setback limits).

d. Floodways determined along channels where active in-steam sand and gravel mining are occurring will require frequent updating and modification.

e. Floodway determinations on alluvial fans are not possible with the present procedures.

f. Floodways are determined by using an equal reduction of overbank conveyance on both sides of the stream. One must ask if the encroachments may cause bend accelerations, super-elevated flows in the bends, increased channel velocities or other aggradation or degradation associated problems. Examination of the plan form, in addition to the channel cross section and profile, is equally important.

g. One must consider a smooth transition into and out of a designed floodway zone so as not to cause upstream or downstream sediment transport problems.

h. Floodway studies only consider the 1-percent chance flood for analysis. Proper planning and design of floodways and flood plain development should consider the effects of the full range of expected flows. A properly designed floodway will operate efficiently and remain stable during low, intermediate and high flows. Remember that 90 percent of the sediment moves during 10 percent of the time.

i. Floodway maintenance is essential. Floodway computations are based on the assumption that the channel's roughness and cross section will remain the same over time. Clearing and snagging of shrubs and debris is essential.

j. If changes in the characteristics of the upstream watershed are possible with respect to discharge or total sediment load, this must be considered in the management of the floodway.

k. The location of the floodway is based on the present location of the channel. One needs to ask:

> Has the channel always been there?
> If not, when and why did it shift locations?
> How far did it move?
> What is the likelihood of it shifting in the future?

l. Can modification of channel storage capacity result in upstream or downstream sediment related problems?

m. Channel blockage during a storm by debris accumulation may lead to increased water surface elevations, channel bank erosion and hydraulic conditions drastically different from those computed with the traditional assumptions.

An assessment of the channel stability can provide a relative sense of the likelihood of having some of the problems listed above. If the fluvial system appears sufficiently stable for application of the rigid-boundary model, then traditional floodway procedures should be used. If the stream assessment indicates a likely channel change that would invalidate the floodway determination, then the following procedures should be considered.

8.7 ALTERNATIVE PROCEDURES

Perhaps the most important concept to realize about fluvial systems and especially alluvial channels is that they are dynamic systems that are constantly attempting to achieve a state of balance or equilibrium. Consequently, alluvial channels are either adjusting to altered conditions or are in a state of dynamic equilibrium with their present conditions. In either case, natural and man-induced changes can initiate responses that may propagate over large areas for many years.

Alternative floodway computation procedures must first determine whether the alluvial channel being analyzed is stable or active and whether the assumptions listed in Section 3.4 are sufficiently met. If the channel is reasonably stable, then traditional procedures can be applied. If not, then the degree of instability and specific dynamic features affecting the floodway should be considered. Options available range from added floodway criteria, which provide a factor of safety consistent with the risk and uncertainty, up to detailed quantitative analysis.

8.7.1 GENERAL SOLUTION APPROACH. The recommended procedure for evaluating alluvial channels involves three levels of analysis. Simons, Li & Associates (1982) define the levels as:

I. <u>qualitative</u>, involving channel classification, examination of historical characteristics and application of geomorphic concepts;

II. <u>quantitative</u>, involving detailed geomorphic concepts and the application of basic quantitative engineering relationships;

III. <u>detailed</u> <u>quantitative</u>, involving sophisticated mathematical modeling procedures.

A qualitative Level I analysis provides information necessary to classify the stability of an alluvial channel, examine its past activity and determine whether alternative computational procedures are necessary for delineating the floodway. The general knowledge obtained at this first Level provides understanding and direction for the Level II and III quantitative analyses, should they be necessary. Additionally, the governing physical processes are usually identified in the general solutions of Levels I and II, allowing proper selection (or development) of a model, if the Level III analysis is performed.

The three level approach provides an efficient and logical method of evaluating complex river problems. The risk of error is minimized because all results and conclusions are cross-linked to the other levels of analysis. The following paragraphs discuss some of the important concepts and procedures suggested for each level of analysis when conducting a floodway evaluation.

8.7.2 LEVEL I - QUALITATIVE GEOMORPHIC ANALYSIS. The qualitative geomorphic analysis relies strongly on common sense and practical experience. Geomorphology is the study of surficial features of the earth and the physical and chemical processes of changing land forms, while fluvial geomorphology is the geomorphology (and mechanics) of watershed and river systems. Qualitative geomorphic techniques are primarily based on well-founded understanding of the physical processes governing watershed and river response.

Therefore, important first steps are to assemble and review previous work and historical data applicable to the study area and to become familiar with the study area. A site visit by key project participants ensures identification of important characteristics of the channel and flood plain. Additionally, being in the study area and contacting the concerned local interest groups provides excellent insight and perspective for the study. Site visits are an essential element of a successful Level I study.

The following is a listing of the type of data that should be collected and examined during Level I analysis. Following a Level I analysis, one should be able to determine whether traditional floodway procedures will be valid for the particular alluvial channel in question.

a. General Channel Slope and Cross Section Characteristics
b. Representative (Dominant) Discharge
c. Bed and Bank Material Characteristics
d. Land-Use and Land-Use Changes
e. Major Structures, Channel Crossings and History
f. Aerial Photographs
g. Flood History
h. Fire History
i. Tectonic Activity
j. History of Bed and Bank Stability
k. Channel Maintenance Records

After completing the necessary site visits, there are a number of simplified concepts and procedures that contribute to a qualitative analysis. These include aerial photograph analysis, evaluation of historical land-use patterns, and application of relatively simple relationships describing basic geomorphic concepts. A Level I analysis may include the following kinds of analyses and data requirements.

a. Collect and review all available reports, maps and data pertaining to the reach.

b. Compare the general characteristics of the alluvial channel to the idealized floodway characteristics. Ask yourself if any of the assumptions listed in Section 2.4 are violated.

c. Examine aerial photographs. Determine plan form characteristics of the river channel and flood plain. Examine historical shifts in the channel and flood plain locations.

d. Classify the channel according to standard geomorphic procedures (Schumm, 1977). Determine whether the channel is straight, meandering or braided, and estimate its relative stability.

e. Using Lane's (1955) relationship, evaluate the general characteristics of the channel and floodway and their responses to various changes in flow, sediment load or channel modifications.

f. Examine the bed and banks. Do they appear to be stable or unstable? Where and why? Locate unstable reaches on your plan and profile maps. Plot historical bank full channel width versus time, and compare the widths to the peak flows experienced during each year.

Based on Level I analysis, the decision should be made whether to apply traditional floodway procedures, consider additional factors like velocities or setback limits, or proceed with Level II analysis. Further analysis may be beyond the scope of the FEMA floodway study.

8.7.3 MINIMUM FLOODWAY SETBACK. Several local agencies, such as The Pima County Transportation and Flood Control District, have adopted alternative flood plain management ordinances to complement traditional floodway determination. A common problem in the arid Southwest is the large scale lateral migration of the channel during flood events. In Pima County, Arizona, strict flood plain management ordinances require that structures be set back a prescribed distance from the river. The distance is based on the type of channel and the magnitude of the expected 1-percent flood discharge. Major watercourses may require a lateral setback of 500 feet, while smaller ephemeral streams, with a 1-percent discharge of less than 3,000 cubic feet per second, may only require a setback of 50 feet.

The intent of the setback ordinance is to provide protection from the unpredictable threat of lateral river migration that occurs on these rivers. The results from a Level I assessment should provide an indication of the historical migration of the stream, and that information could be used to determine the minimum setback requirements. The traditionally computed floodway would be adjusted, if required, to provide the minimum setback to the fringe area.

Alternative methods of providing additional flood plain management and protection should be encouraged as part of the floodway determination process until more accurate methods are available.

8.7.4 MAXIMUM VELOCITY. The maximum velocity, or a change in velocity, should be an added evaluation consideration for floodways on alluvial streams. If the floodway computation indicates an increase in the channel velocity, it is likely that velocity increase will cause a change in the alluvial channel. The channel velocity is usually computed for both the existing and floodway conditions; therefore, it can be compared. A change in the channel velocity of more than 10-percent may be an indication that the floodway is too restrictive. However, whether the increased channel velocity is too great depends on the channel bed material.

The maximum permissible velocity recommended for channel design could be used for evaluating floodway channel velocity. Tables and guidelines can be found in Chow, paragraph 7.9 (1959), USACE EM 1110-2-1601, paragraph 13 (1970), and Simons, Li & Associates, paragraph 12.2.2 (1982). Much of the data were developed for irrigation canal design with steady flow conditions; however, the values will provide an indication of the "safe" velocity for various bed materials.

8.8 SUGGESTED FLOODWAY PROCEDURES

Alluvial Fans - While new mathematical models are being developed to simulate flooding on an unbounded plain (Hamilton, Schamber,& MacArthur, 1987), these procedures are still in the developmental stage. The continued development and testing of models is encouraged. However, until the procedures and computer programs are in general use, the current Guideline procedures (FEMA, Sep. 1985) should be used for general studies on alluvial fans.

Braided Streams - The lack of stability of the various stream paths in a braided stream make it very difficult to consider them separately for floodway definition. The general case should consider the braided streams as a whole channel. Under the Guidelines, floodway definition must be outside the channel; therefore, all of the braided streams would be in the floodway. Additionally, the suggested procedures for alluvial streams should be followed.

Alluvial Streams - The assessment of the study reach is essential in determining the proper study procedure. Level I Qualitative Geomorphic Analysis is recommended; therefore, the selection criteria for a study contractor should include an evaluation of their training and experience with alluvial streams. The value of a Level I assessment is dependent on the experience of the person performing the analysis. An assessment requires considerable engineering judgment.

Section 8.1 presented three considerations for applying the floodway procedures. The first, the applicability of the "ideal floodway" model, is an engineering decision that can be made prior to an assessment of channel stability. The second two depend on an assessment.

The validity of the computed water surface elevations and resulting conveyance and velocities cannot be determined with the rigid-boundary model approach. However, sensitivity of the computed values can be determined by modifying the Manning's "n" values to reflect changes in bed form and by modifying cross sections to reflect debris accumulation. If an assessment determines that these types of changes are likely, then an attempt to quantify and evaluate the impacts should be made. Section 7.3 provides suggestions and references.

Lateral Migration, Bank Instability and Erosion Hazards - Concern for the future of the floodway, and for the development that would be allowed in the fringe, is the motivation for the third question: Is the study reach "reasonably" stable? Again, the Level I assessment should provide some indication of relative channel stability. From the available information using the current floodway computation procedures, the channel and overbank velocities and the setback distance of the floodway line should be evaluated. The change in velocities under proposed floodway conditions should be reviewed for the impact on channel stability. Section 8.7.4 provides references for permissible channel design velocities. The stream assessment should provide sufficient information for defining reasonable velocity limits.

The use of minimum setbacks is a protective measure reflecting the potential for the stream to shift into the fringe area. The history of the stream, from the stream assessment, is the best indicator for future expectation. Present math models to predict lateral migration are developing, but are not practical for general floodway analysis. In the absence of a local setback standard, the historic stream migration should be considered when setting the floodway limits. A minimum setback should be included as a safety factor to account for the uncertainty in the engineering procedures used.

9. HIGH VELOCITY STREAMS

In terms of human hazard, high velocity in streams could include velocities greater than five feet per second. Some may place the value near two feet per second (Muller, 1975). When velocities exceed two feet per second and depths are greater than two feet, there is a potential for sweeping cars off roads. With greater depths, it is difficult for an adult to maintain stability while walking.

For sediment transport, the velocity that causes motion of the bed material (incipient motion) may be considered the threshold for high velocity. Using average velocity information from stable channel design, velocities from two to six feet per second are suggested for many common bed materials. Therefore, velocities greater than six feet per second can often be considered a problem for maintaining existing channel form and for protecting any floodway fringe fill.

For flow stability, the velocity near critical velocity would be the point of concern. Most likely, the critical velocity would be higher than the velocities stated above. Flows at or near critical depth are generally unstable (Simons, Li, & Assoc., 1982). Near critical depth, any changes in flow conditions can result in major changes in flow regime and depth. For this reason, flow depths near critical depth are discouraged in channel design.

The velocities in the supercritical flow regime are even higher than those listed above. In this state, the inertial forces (velocity) are dominant. The flow is often described as rapid, shooting and torrential (Chow, 1959). Under this condition, gravity waves created by disturbances in the channel are swept downstream. The computation of water surface profiles must start at the upstream end of the reach and proceed downstream because the impact of changes in the cross section is carried downstream.

9.1 APPLICABILITY OF FLOODWAY CONCEPT

The applicability of the floodway concept should be considered separately for subcritical and supercritical flow regimes, because the impact of development on the water surface elevation is different. When computing water surface profiles, subcritical flow is usually assumed because it is the most common flow regime in flood plain studies. Also, the water surface profile computer programs have internal checks to ensure that the profile is in the proper flow regime. However, it is possible to have both flow regimes in compound cross sections, i.e., supercritical regime in the channel and subcritical flow in the overbanks (Schoellhamer, Peters, & Larock, 1985).

9.1.1 SUPERCRITICAL FLOW. Under supercritical flow conditions, the velocity head may be equal to or greater than the depth of flow. Any obstruction of flow can result in a standing wave, with no effect on the flow upstream. Channel contractions can develop disturbances at the walls of the channel, which form standing waves reflected diagonally from wall to wall in the downstream direction. Channel expansions can cause a hydraulic jump, which is unstable in both location and height (Simons, Li, & Assoc., 1982).

Supercritical flow is a very challenging computational problem, as well as a very hazardous condition for flood plain development and use. Flood plain development can cause the water surface elevation to increase or decrease, depending on the situation. Given the reduced significance of the flow depth to the total energy of flow, it is obvious that using the change in water surface elevation as an evaluation criterion may not properly reflect the impact of development under supercritical flow conditions. The total energy elevation would be a better choice. The question remains whether floodway encroachments should be computed at all.

If the flow is primarily confined to the channel, then there should not be any flood plain encroachment. The existing flood plain should also be considered the floodway. The potential impact from imposed flow contractions and expansions would be sufficient justification for this policy. However, when the added concern for human safety is evaluated against the high velocities associated with supercritical flow, there is little justification for development within the existing flood plain.

With flood flow in the overbank areas, it is possible to have a supercritical flow condition indicated and still not have hazardous velocities and depths in the overbank area. Under this condition, encroachment computations may seem appropriate. To avoid the confusion from computed encroachments causing the water surface elevation to both increase and decrease along the study reach, the energy elevation is recommended as a better evaluation parameter. Also, the actual water surface elevation at the flood plain fringe may be closer to the computed energy elevation for the cross section. A consideration of overbank velocity is also recommended to ensure that velocities are reasonable if development should occur.

9.1.2 SUBCRITICAL FLOW. If flow depths are near critical depth, then the floodway application problem is similar to that for supercritical flow. Modifications to the cross sections can produce erratic results. The preceding discussion on supercritical flow would generally apply to the near critical depth condition. The problem is defining what constitutes near critical depth.

Various studies have been performed to determine the general unstable zone (USACE, 1970). A range of unstable depths is between 1.10 times critical depth and 0.9 times critical depth.

Using the Froude number (F), the range is between 1.13 and 0.86. While the Froude number is an obvious choice for defining near critical depth, it may not be readily available in the output of the computer program used to compute the water surface profiles. An approximate equivalent can be developed from the ratio of velocity head to depth. For stable subcritical flow, the velocity head (HV) should be less than one-third the hydraulic depth (D) ($HV/D < 0.33$). If the velocity head is greater than one-third the hydraulic depth, the flow would be considered potentially unstable.

The impact of development in a flood plain with high velocity flow is more pronounced than with lower velocities. The floodway concept is applicable to high velocity, subcritical flow streams. However, for flood plain studies with subcritical flow in the unstable range, the floodway computation should be based on energy elevation. For subcritical flow that is sufficiently above critical depth, the current floodway procedure is appropriate.

9.2 FLOODWAY COMPUTATION PROBLEMS

Ignoring the concerns about sediment transport, the primary computational problems with high velocity flow come from the unstable conditions near and below critical depth. As flow depths approach critical depth, the velocity head becomes a more significant portion of the total energy. The velocity head is equal to half of the hydraulic depth when flow is at critical depth in a rectangular section (Chow, 1959). As previously described, flow conditions become unstable when the velocity head is greater than one-third the hydraulic depth.

As the flow depth decreases below critical depth, the velocity head continues to constitute an increasing proportion of the total energy of the flow. The computation of water surface profiles and floodways generally becomes more difficult with increasing velocity of flow. The computed profiles are generally more sensitive to variations in input data. Floodway computation problems for supercritical and subcritical flow regimes are discussed below.

9.2.1 SUPERCRITICAL FLOW. Computing water surface profiles for supercritical flow is more difficult than for subcritical flow. The computation process is more sensitive to the quantity and accuracy of the data provided. Generally, the greater the velocities, the more cross-sectional data required. Also, variations in the loss coefficients and number of cross sections have a greater impact on the computed energy loss and resulting water surface profile. Adding the analysis of encroachments to the basic modeling difficulties compounds the analysis problems.

Floodways should not be computed if there is little or no overbank area. If there are isolated portions of the flood plain that appear reasonable for development, then those locations should be evaluated. The primary evaluation should be based on velocity and depth. The potential areas can be blocked out in the model, and new water surface profiles computed. Again, the change in energy elevation should be evaluated along with the change in water surface elevation. Neither should exceed one foot, or applicable lower state or local standards.

When there is sufficient overbank flow to consider the standard floodway computation, there may be some additional computational considerations. A study at the Hydrologic Engineering Center developed a method for computing a subdivision Froude number for compound sections (Schoellhamer, Peters, & Larock, 1985). A study conclusion was that the method could help identify mixed flow conditions that would violate the assumptions of the standard step method. For shallow overbank depths, an imaginary subdivision Froude number was computed in some of the cases. This would indicate that the computation process may not be valid because there exist two different flow regimes, and the one-dimensional approach cannot handle both. Under this situation, it may appear that encroachments would be feasible, but the model may not be appropriate.

9.2.2 SUBCRITICAL FLOW. The degree of computational difficulty varies, but it generally is greatest near critical depth and tends to decrease as the flow depth increases above critical depth. For subcritical flow, the problem area is the unstable zone with a Froude number greater than 0.86, or a depth below 1.1 times critical depth. These limits are approximate "rule-of-thumb" values to indicate where the more serious problems occur. There still may be computational difficulties for streams at greater depths but with higher velocities.

For higher velocity conditions, the redistribution of flow, resulting from eliminating conveyance in the flood plain fringe, has a greater impact on the computed velocity head and energy losses. While the computed profile may remain subcritical, the change in water surface elevation due to the flood plain encroachment is more uneven. The water surface elevation may even decrease in a location, due to the encroachment. In the more extreme cases, the encroachment may cause critical depth to occur. Usually the next upstream cross section will have a much-larger-than-expected increase in water surface elevation. To correct the situation, the encroachment must be reduced at the section with the decrease in water surface elevation, and the profile computed again. This process may have to be repeated many times to provide a reasonable balance between the encroachments and the resulting water surface profile.

While the floodway concept is considered applicable to high velocity subcritical streams, the computation of a floodway is definitely more difficult. The difficulty is compounded when the model data lacks sufficient cross sections and has not been thoroughly calibrated. When there is computational difficulty and a balanced solution for water surface elevation cannot be obtained, the HEC-2 program may assume critical depth. In some cases the true profile may be near critical depth, but often the true solution is well above this depth. If these computational problems are not eliminated before the floodway computations begin, the resulting computations will yield erratic results. While it may be tempting to say the floodway concept does not apply in this case, the fact is that the model is not sufficiently developed to compute a floodway.

9.3 SUGGESTED APPROACH

With the significant differences between supercritical and subcritical flow, the floodway procedures should be defined separately for the two flow regimes.

9.3.1 SUPERCRITICAL FLOW. The flood plain, as defined by the base flood water surface profile, should be treated as the floodway when there is little or no overbank area. With steep streams in incised channels, there may be some uncertainty about the distinction between channel and overbank. If the cross-sectional shape does not clearly divide into channel and overbank, then cross-sectional areas with depths less than three feet and flow velocities less than three feet per second can be evaluated as overbank areas.

If there is overbank area, the floodway can be delineated by blocking the conveyance along the fringe that contains flow less than three feet deep and has a velocity less than three feet per second. It is unlikely these areas contain supercritical flow. The floodway water surface profile would then be computed, and both the total energy and water surface elevations would be evaluated against the appropriate criterion for change in elevation. If either change exceeds the criterion, the encroachments would be reduced until the results were within the maximum allowable increase.

The total energy elevation is recommended as the operational base flood elevation for supercritical streams. The true water surface elevation will be close to the energy elevation along the fringe of the flood plain. Also, the total energy elevation is a better indicator of the flood risk in the flood plain. The water surface elevation will tend to rise to the energy elevation if the flow is obstructed. This is consistent with the present Guidelines (FEMA, Sep. 1985, pg.2-12).

9.3.2 SUBCRITICAL FLOW. The conveyance-based floodway computations are not practical when the base flood water surface elevation is at or near critical depth. Critical depth solutions may appear in high velocity water surface profile computations because the model is not adequate. Therefore, study contractors should be encouraged to improve the model, by adding cross sections and other refinements, in order to eliminate assumed critical depth solutions.

If the base flood profile is at, or below, 1.1 times critical water surface depth for numerous cross sections, then the procedures for supercritical flow are recommended. If the base flood profile is well above critical depth, the current floodway procedure is recommended. However, the use of total energy elevation should be freely allowed for high velocity streams that present floodway computational problems.

The change in energy or water surface elevation should not be the only criterion. Given the potential hazards of flow depths and velocity, as well as the floodway profile response to any changes in cross sections, the floodway evaluation should also consider velocity and depth in the floodway fringe. While this is not required by the Guidelines, it is encouraged as an added consideration for high velocity streams. Maximum values of 3-feet of depth and 3-feet per second velocity are recommended for supplemental evaluation.

10. DEVELOPED FLOODWAY

Existing development in the floodway area that has been defined by current floodway procedures is common. Much of the urban growth of this country has occurred on the flood plains. In spite of the billions invested in flood control works, the national cost from flood damage continues to rise. Flood plain development and the resulting increase in flood damage led to the creation of the National Flood Insurance Program to promote land use planning.

Most of the flood insurance areas are studied because there is, or soon will be, flood plain development. In many cases the development has been so extensive that the potential floodway area is seen as an essential part of the community. The demand for the land may have driven up the value of the property to a level that makes any control of the land use very difficult for local communities to accept. The floodway impact to the property value could be great. Therefore, any procedure used must be as reasonable and equitable as possible.

Developed floodway application problems may apply to any of the floodway situations reviewed in the previous chapters. Floodway development is often just one more added complication to any of the previous study problems. However, the purpose of the floodway must be kept in mind while considering the problem and alternative solutions. The floodway is required to provide safe passage of the base flood and to limit additional development so that it has an insignificant impact on the flood elevation.

10.1 FLOODWAY COMPUTATION PROBLEMS

The computation of water surface profiles is more difficult with buildings and other constructed obstructions in the flood plain. There are two basic approaches that can be used to model flood plain development in this situation: either model the buildings, or adjust the loss coefficients to reflect the added obstructions.

10.1.1 MODELING BUILDINGS. Modeling the individual structures on the flood plain may require additional cross sections. Four cross sections may be required to show a blocked portion of the flood plain, such as the presence of a building. (see Figure 10.1). Two cross sections describe the downstream and upstream faces of the building, and two sections describe the open area downstream and upstream from the building. If there are many buildings to define, the number of cross sections could become large. Also, defining all the buildings could become quite tedious.

If the flood plain development is fairly complete and the structures are approximately in a line, then the row of buildings could be treated as a continuous zone that is obstructed for the length of a city block. (With the usual zoning setback requirements for a lot, this would be common.) This approach would assume that there was no flow between the individual buildings. While this would reduce the number of cross sections required, it would also indicate lower energy losses than a model that included all the contraction and expansion of flow between the buildings. The loss coefficients could be adjusted higher to account for the simplified model's lower computed losses.

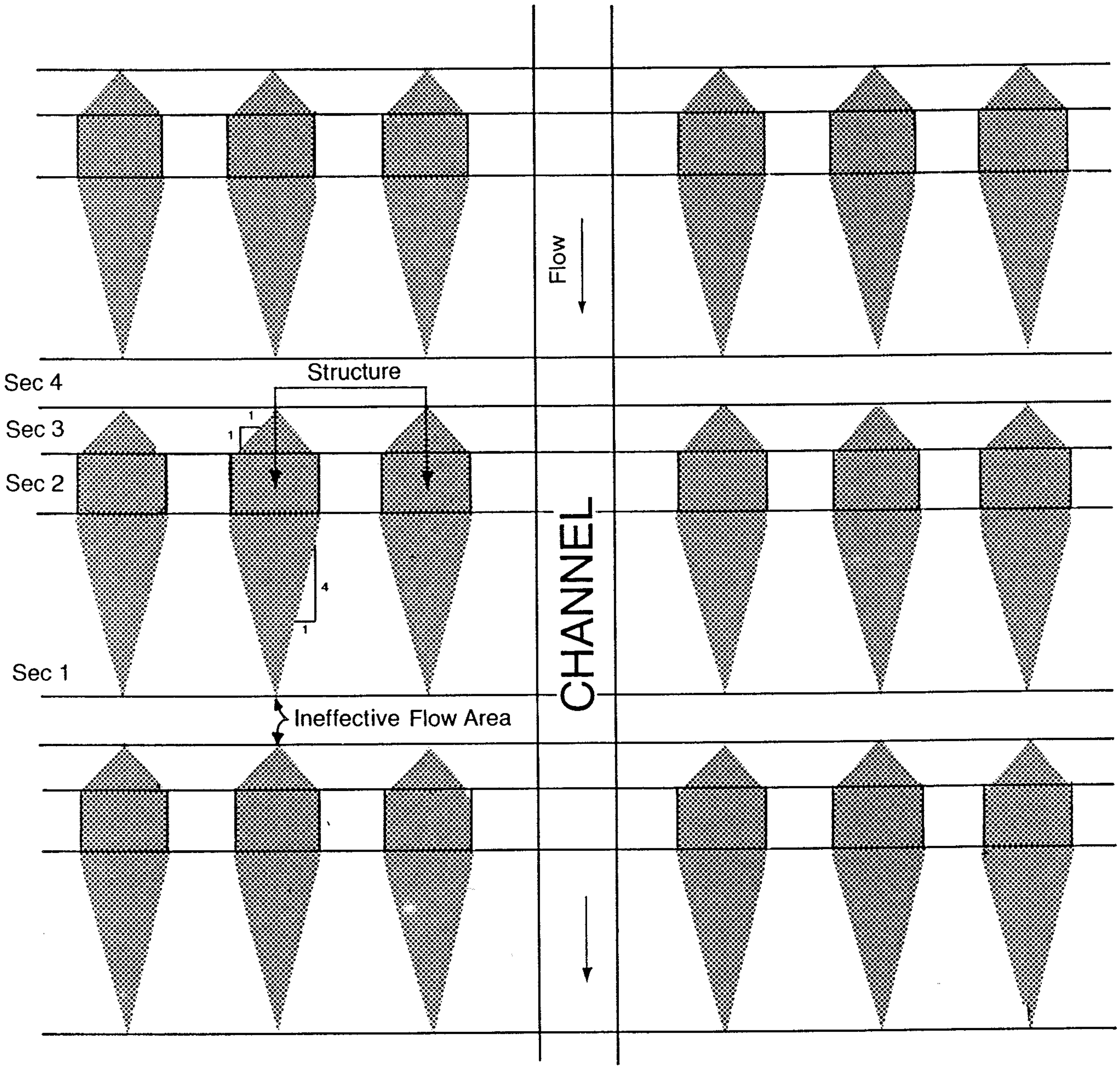


Figure 10.1 Cross Section Location and Effective Flow Boundaries (from Montalvo, 1981)

Regardless of the approach, the flood plain model will require more cross-sectional data than it would if the development were not there. Even with the added data, the complicated potential interflow that would occur through side streets would generally be ignored. The complicated flow paths cannot be modeled with a one-dimensional model.

The main advantage of modeling the buildings is that the flood plain model is more physically based. The model can be used to assess the impact of additional construction in the flood plain. The disadvantage is that added cross-sectional data will increase study costs.

10.1.2 ADJUSTING LOSS COEFFICIENTS. Instead of modeling the buildings in the flood plain, the impact of the obstructions can be estimated by adjusting the Manning's "n" value to reflect the presence of buildings. A procedure was developed by the U.S. Geological Survey (USGS) to estimate the adjusted "n" value based on the relative proportion of blocked cross-sectional width and the relative proportion of open space between the buildings (Hejl, 1977). Water surface profiles were computed, modeling the buildings in the manner described in Section 10.11. The model was based on a uniform density and spacing of the buildings. An equation to adjust the Manning's value was then developed based on the computed water surface profiles.

The advantage of using the adjusted Manning's "n" value is the relative ease of application. The flood plain model is basically the same as for non-urbanized areas. An "n" value is estimated, based on the available flow paths, and then adjusted, based on the relative obstruction of the development. The disadvantage of this approach is the inability to evaluate incremental changes to the flood plain development. The computed water surface profile is based on the average development.

10.1.3 BRIDGE CROSSINGS. A common floodway computation problem is defining floodways in the vicinity of bridges. The modeling of flow through bridges is complex. It may account for a major portion of the modeling effort, after the basic data are prepared for the program. The problem considered here is not how to compute flow through bridges, but how to compute and interpret the computed floodway.

The floodway computation problems usually occur when the bridge model indicates that the base flood discharge will pass under the bridge, either as **low flow or pressure flow.** This situation usually means that the flood plain flow must contract to pass under the available bridge opening. Once past the bridge, the flow expands to the limits of the flood plain.

To model the contraction and expansion of flow, the cross sections must be adjusted to define only the available conveyance area in the immediate vicinity of the bridge. This often means that the cross sections upstream and downstream from the bridge do not define any conveyance area in the overbanks because the abutments of the bridge fill the overbank area.

The floodway computations are based on reducing overbank conveyance in order to reflect development blocking the flow in the fringe area. However, at the cross sections in the vicinity of the bridge, there is no conveyance in the overbanks. The computed floodway encroachment would be defined at the edge of the bridge opening. If the computed encroachment locations are used as the floodway limits, the floodway would contract and expand through the bridge just like the flow.

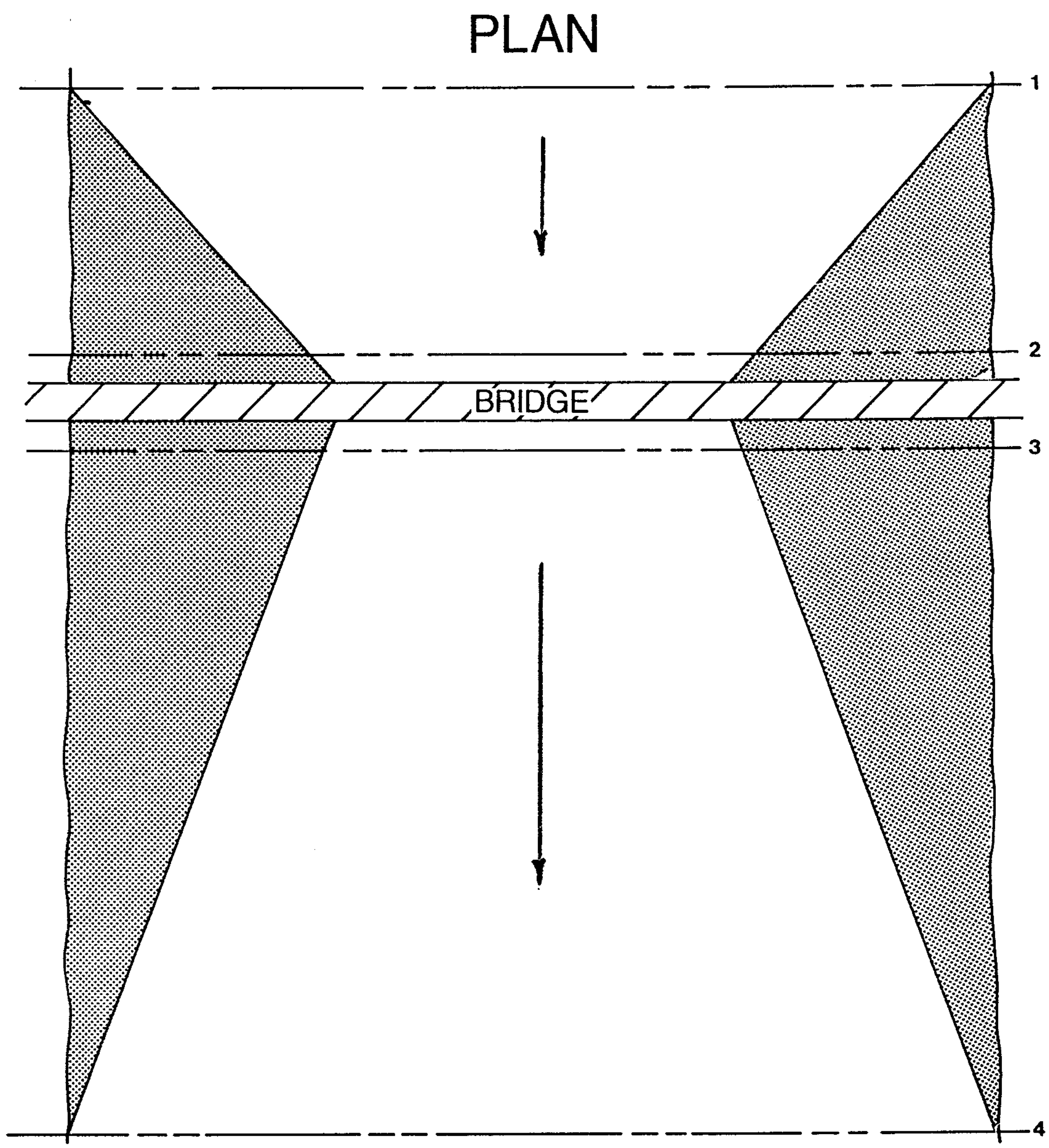


Figure 10.2 Effective Flow in the Vicinity of Bridges

The simple solution to this problem is to use the computed floodway limits from the full-flow cross sections, upstream and downstream from the bridge in order to define the floodway through the vicinity of the bridge. The conveyance-based encroachment computations (e.g., Methods 3 and 4 in HEC-2) are meaningless when there is no conveyance available in the overbank area.

If the base flood also flows over the bridge and/or roadway approaches, the overflow is usually modeled as **weir flow.** The conveyance in the overbank areas immediately upstream and downstream from the bridge is usually considered effective, on the assumption that the overbank flow can pass over the bridge. Therefore, the conveyance-based encroachment calculation can be performed at the cross sections in the vicinity of the bridge. To be consistent, the bridge overflow should also be limited to the floodway width computed for the upstream and downstream cross sections. The HEC-2 program provides the floodway width limitation to the weir flow calculation, if requested by adding a value of 0.01 to the encroachment option input data.

10.2 FLOODWAY APPLICATION PROBLEMS

The major problem here is that construction already exists in the flood plain, and perhaps in the potential floodway. There is a difference in application between rural and urban development. In the rural area, with low density development, the problem of floodway development may not be too severe. Existing development probably would not have had a significant impact on the base flood elevation. The traditional floodway computation could be used to define the limits of development. However, there may be application problems associated with limited development in wide flood plains, as discussed in Section 6.3.

In the urban environment, high density development probably exists in the flood plain and maybe in the potential floodway. When the floodway is computed, the present assumption is that the future development will block the conveyance along the edge of the flood plain. However, that area may already be largely developed. There may be very little development area remaining.

The floodway evaluation is based on the impact of development on the water surface profile. However, it is difficult to model the developed flood plain, and the impact of additional development. There may be some flood conveyance area along the streets and between buildings with the present flood plain development. Along the flood plain fringe, should that conveyance area be considered totally blocked for floodway determinations? Assuming that the fringe is totally blocked may not be applicable when only a few remaining lots are available for development.

The water surface profile is already higher than the natural condition profile due to existing development. Adopting a floodway that could ultimately raise the base flood elevation one foot above existing flood levels is a significant application problem. Some states have set a lower maximum change in water surface elevation in recognition of the impact of existing development (Goddard, 1978).

10.3 SUGGESTED APPROACH

To support the goals of the floodway concept, there needs to be a computational procedure that reasonably evaluates the impact of flood plain development on the base flood elevation. Given the constraints of time and funds for floodway studies, the procedure should not be too complicated. Of the procedures reviewed, a combination of traditional floodway computation and simplified urban modeling is considered the most practical solution to the problems identified.

The buildings on the flood plain should be included in the cross-sectional data if they are few in number, or if they can be treated as a block development. A row of buildings can be defined with two cross sections; therefore, the increased input requirements are relatively small. This approach is preferable because it is more physically based.

The use of adjusted Manning's "n" values to model the flood plain development is recommended for extensive development. The flood plain model would be developed without detailed modeling of obstructions caused by buildings. The roughness coefficients would be estimated to include the buildings. While this may not be the "best possible solution," it would provide base flood elevations and a model that could be used to compute a floodway. The maximum allowable change in water surface elevation would be the appropriate value for the study area. The primary disadvantage of this procedure is that the model cannot be used to evaluate the impact of a local modification or additional development proposal.

The floodway would be computed using the current procedures. Because the "n" values reflect the existing development, the average impact of existing development in the floodway would be considered. Existing development in the floodway would be subject to the current limitations on substantial changes or additions. Ideally, the existing development in the floodway should be removed.

Establishing an alternative floodway approach for the developed flood plain would create administrative problems. The basis for determining when a flood plain is fully or near fully developed would require arbitrary rules that may be subject to continued debate. Even though the existing development may be extensive and floodway computation may not seem that logical, the floodway computation is recommended.

The recommended floodway computation is not difficult. The use of "n" values to model extensive development is currently used in some flood plain studies. The computed floodway would still provide a basis for limiting further development in the zone most needed for flood conveyance. The floodway also provides for added development in the fringe. Because the floodway computations would assume complete loss of conveyance in the fringe, the floodway determination should be conservative, even if the one-foot maximum change in elevation is used.

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APPENDIX A

CALCULATION OF ENCROACHMENT STATIONS BY METHOD 4

A. CALCULATION OF ENCROACHMENT STATIONS BY METHOD 4

The following "hand-calculation" example is provided to illustrate the HEC-2 Method 4 procedure for determining encroachment stations for a cross section. Encroachment Methods 5 and 6 start with the Method 4 calculation and Method 3 uses the same encroachment station computation, given a conveyance reduction value. The HEC-2 output for the example is also shown to allow comparison of results and to relate the printed output to the basic computations.

First, the conveyance and water surface elevation is computed for the natural conditions. These computations are made in the first profile of a multiple profile run using HEC-2. On a second, or subsequent profile, the water surface elevation is increased an incremental amount, as specified on the ET record input. (For example, an ET value of 10.4 indicates an increase of 10-tenths, one-foot, using Method 4.) The conveyance, at the increased water surface elevation, and the conveyance increase over existing conditions are computed. Then the ratio of conveyance increase to the conveyance at the higher elevation is computed. The conveyance increase ratio becomes the TARGET for computing the encroachment stations.

The encroachment stations are computed based on removing half of the conveyance increase (TARGET) from each of the two overbank areas (equal conveyance reduction). However, encroachments will not go inside the bank stations. If the conveyance in one of the overbanks is less than half of the TARGET, the overbank area is removed (encroachment station equals bank station) and the remaining portion of the TARGET is added to the ratio for the other overbank area.

Starting with the left overbank, the incremental conveyance is accumulated for each ground station until half of the TARGET reduction is bracketed by two stations. The encroachment station is then interpolated between the two stations. The original encroachment station calculation was based on linear interpolation between the ground stations. However conveyance does not varying linearly with distance, so the interpolation was modified to a parabolic equation around 1979. The right overbank encroachment is computed in a similar fashion, working from the farthest right station toward the right bank station. The following example illustrates the computations.

METHOD 4 EXAMPLE

Determine location of encroachment so that conveyance, with a one foot higher water surface elevation, is equal to conveyance for the lower (existing condition) water surface elevation.

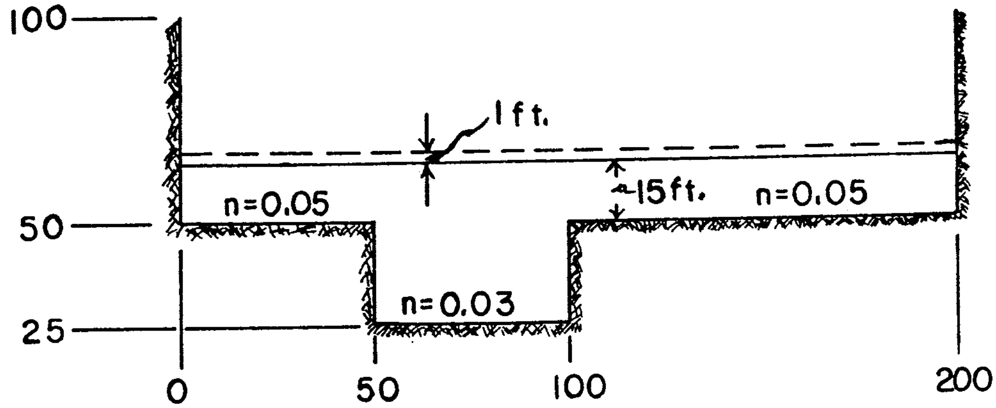

Figure A-1. Method 4 Example Cross Section

Conveyances for natural water surface elevation, NAT K

$K = (1.486/n) * A * R^{2/3}$

$K_{LOB} = (1.486/0.05)*(15*50)*(15*50/65)^{2/3} = 113{,}817$ cfs

$K_{CH} = (1.486/0.03)*(40*50)*(40*50/100)^{2/3} = 729{,}929$ cfs

$K_{ROB} = (1.486/0.05)*(15*100)*(15*100/115)^{2/3} = 247{,}022$ cfs

NAT $K_{TOTAL} = 1{,}090{,}768$ cfs

Conveyances for one foot higher water surface elevation, ENC K

$K_{LOB} = (1.486/0.05)*(16*50)*(16*50/66)^{2/3} = 125{,}459$ cfs

$K_{CH} = (1.486/0.03)*(41*50)*(41*50/100)^{2/3} = 760{,}596$ cfs

$K_{ROB} = (1.486/0.05)*(16*100)*(16*100/116)^{2/3} = 273{,}491$ cfs

ENC $K_{TOTAL} = 1{,}159{,}546$ cfs

Ratio of required conveyance reduction, TARGET

TARGET = (ENC K_T - NAT K_T) / ENC K_T

TARGET = (1,159,546 - 1,090,768) / 1,159,546 = 0.0593

Overbanks reduction ratio = 0.0593 / 2 = 0.02965

Ratios of conveyance at the higher water surface elevation

RATIO LOB = 125,459 / 1,159,546 = 0.1082

RATIO CH = 760,596 / 1,159,546 = 0.6559

RATIO ROB = 273,491 / 1,159,546 = 0.2359

Therefore, there is sufficient conveyance for encroachment calculation. (0.1082 & 0.2359 > 0.02965)

THE FOLLOWING COMPUTATIONS ARE FOR THE LEFT ENCROACHMENT, STENCL

Determine overbank conveyance reduction, K_{OBR}

$K_{OBR} = K_{TOTAL} * (TARGET / 2)$

K_{OBR} = 1,159,546 * 0.02965 = 34,380 cfs

Determine remaining overbank conveyance, REMKX

REMKX = $K_{LOB} - K_{OBR}$

REMKX = 125,459 - 34,380 = 91,079 cfs

Determine conveyance in the cross-section increment that brackets the encroachment target (In this case it is the entire overbank.)

REMK1 = K_{LOB} = 125,459 cfs

Determine the conveyance for half of the cross-section increment

REMK = (1.486/n) * A * $R^{2/3}$

REMK = (1.486/0.05)*(16*25)*$(16*25/41)^{2/3}$ = 54,278 cfs

Determine parabolic equation coefficients, $aX^2 + bX + c = 0$

a = 2 * (REMK1 - (2 * REMK))

a = 2 * (125,459 - (2 * 54,278)) = 33,806

b = (4 * REMK) - REMK1

b = (4 * 54,278) - 125,459 = 91,653

c = - REMKX = -91,079

Determine encroachment stations

Where:

XLEN = Distance between the two bracketing stations
ST = The right bracketing station
XST = The left bracketing station
XL = The encroachment distance from XST
X = The ratio (XL / XLEN)

For the left overbank encroachment:

ST = 50 feet
XST = 0 feet
XLEN = 50 - 0 = 50 feet

$$X = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

$$X = \frac{-91,653 + \sqrt{91,653^2 - 4 * 33,806 * -91,079}}{2 * 33,806}$$

X = (-91,653 + 143,932) / 67,612 = 0.773

STENCL = ST - (X * XLEN)

STENCL = 50 - (0.773 * 50) = 11.339 feet

The right encroachment station is computed in a similar fashion. The cross-section data used in this example was entered into HEC-2 as cross-section number 1.00, with a discharge of 1000 cfs. The following output can be compared to the hand calculation. **Note that HEC-2 uses an indication discharge (Q1) as Conveyance (K). Q1 = 0.01 * K**

```
SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG      HV      HL      OLOSS   BANK ELEV
Q        QLOB     QCH      QROB     ALOB     ACH     AROB    VOL     TWA     LEFT/RIGHT
TIME     VLOB     VCH      VROB     XNL      XNCH    XNR     WTN     ELMIN     SSTA
SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC     ICONT   CORAR   TOPWID    ENDST


*PROF 2

*SECNO 1.000
 2800 NAT Q1=  10908.72  WSEL=     65.00  ENC Q1=  10908.72  WSEL=     66.00  RATIO=   .0000
 NAT Q1=  11597.  RATIOS LOB,CH,ROB=   .1082   .6559   .2359  WSEL=     66.00

3470 ENCROACHMENT STATIONS=      11.3      189.0  TYPE=     4   TARGET=        .059
    1.00    41.00    66.00      .00    65.00    66.00      .00      .00      .00    50.00
   1000.      85.     695.     220.     619.    2050.    1425.       0.       0.    50.00
     .00      .14      .34      .15     .050     .030     .050     .000    25.00    11.34
 .000001       0.       0.       0.        0        0        0      .00   177.70   189.04
```

Figure A-2. HEC-2 OUTPUT FOR METHOD 4 EXAMPLE CROSS SECTION

Appendix B HEC-2 FLOODWAY EXAMPLE - FIRST TRIAL

```
*******************************************************          ****************************************
*  WATER SURFACE PROFILES                             *          *  U.S. ARMY CORPS OF ENGINEERS        *
*  VERSION OF NOVEMBER 1976                           *          *  THE HYDROLOGIC ENGINEERING CENTER   *
*  EXPERIMENTAL VERSION OF 1988 RELEASE               *          *  609 SECOND STREET, SUITE D          *
*                                                     *          *  DAVIS, CALIFORNIA 95616             *
*  RUN DATE     1/ 4/88   TIME   10:13: 0             *          *  (916) 756-1104 (FTS) 460-1748       *
*******************************************************          ****************************************


                                     X     X  XXXXXXX   XXXXX             XXXXX
                                     X     X  X        X     X           X     X
                                     X     X  X        X                       X
                                     XXXXXXX  XXXX     X         XXXXX    XXXXX
                                     X     X  X        X                 X
                                     X     X  X        X     X           X
                                     X     X  XXXXXXX   XXXXX            XXXXXXX
END OF BANNER
```

```
                                                                            THIS RUN EXECUTED 04JAN88   10:13:00
**************************************************
 1988 TRIAL VERSION OF HEC2
**************************************************


T1   FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2   First Profile is Existing Condition; Method 4 with 0.8 & 1.0 in 2nd & 3rd
T3      COW CREEK NEAR PALO CEDRO

        * * Remarks Indicate Added Encroachment Input * *
   INQ (J1.2) specifies field to read on ET and QT
   Second field of ET = 0 indicating no encroachment

J1  ICHECK    INQ      NINV     IDIR     STRT     METRIC   HVINS    Q        WSEL     FQ

       0.       2.       0.       0.   .000000     .00      .0        0.   392.000    .000


   J2.10 = 15 requesting Flow Distribution for first profile

J2  NPROF     IPLOT    PRFVS    XSECV    XSECH    FN       ALLDC    IBW      CHNIM    ITRACE

     1.000     .000   -1.000     .000     .000     .000     .000     .000     .000   15.000


   Summary tables 110 and 200 are Pre-defined Floodway Data Tables

J3  VARIABLE CODES FOR SUMMARY PRINTOUT

    110.000  200.000     .000     .000     .000     .000     .000     .000     .000     .000


NC        .050      .050      .045      .100      .300      .000      .000      .000      .000      .000

   1-percent Chance Flood Discharge is defined on all QT fields to be read

QT       3.000  53000.000  53000.000  53000.000   .000      .000      .000      .000      .000      .000

   ET.2 = 0 indicates No Encroachment on first profile (J2.2 = 2)
   ET.3 = 8.4  "  0.8 foot rise using Method 4 when J2.2 = 3 in second profile
   ET.4 = 10.4 "  1.0 foot rise using Method 4 when J2.2 = 4 in third profile

ET        .000      .000     8.400    10.400      .000      .000      .000      .000      .000      .000

   No additional ET records required if no change in scale or method desired.
```

X1	.080	21.000	300.000	530.000	.000	.000	.000	.000	.000	.000
GR	410.000	.000	400.000	40.000	390.000	300.000	380.000	330.000	372.000	350.000
GR	368.000	360.000	368.000	480.000	372.000	490.000	380.000	520.000	390.000	530.000
GR	395.000	700.000	390.000	820.000	380.000	870.000	375.000	890.000	380.000	940.000
GR	390.000	960.000	395.000	1000.000	390.000	1100.000	390.000	1160.000	400.000	1200.000
GR	410.000	1500.000	.000	.000	.000	.000	.000	.000	.000	.000
X1	.210	18.000	150.000	390.000	750.000	750.000	800.000	.000	.000	.000
GR	410.000	.000	400.000	130.000	390.000	150.000	380.000	175.000	373.000	200.000
GR	369.000	210.000	369.000	300.000	373.000	320.000	380.000	380.000	390.000	390.000
GR	395.000	600.000	390.000	680.000	385.000	900.000	390.000	980.000	400.000	1040.000
GR	403.000	1100.000	395.000	1400.000	400.000	1600.000	.000	.000	.000	.000
X1	.340	16.000	50.000	275.000	650.000	690.000	670.000	.000	.000	.000
GR	410.000	.000	390.000	50.000	373.500	100.000	370.000	110.000	370.000	210.000
GR	373.500	220.000	380.000	250.000	390.000	275.000	395.000	500.000	395.000	1500.000
GR	390.000	1720.000	390.000	1780.000	400.000	1820.000	403.000	2200.000	398.000	2900.000
GR	400.000	3300.000	.000	.000	.000	.000	.000	.000	.000	.000
X1	.550	13.000	20.000	350.000	1100.000	1100.000	1100.000	.000	.000	.000
GR	410.000	.000	390.000	20.000	374.500	90.000	371.500	100.000	371.500	200.000
GR	374.500	210.000	380.000	280.000	390.000	350.000	395.000	600.000	395.000	1200.000
GR	400.000	2200.000	405.000	2350.000	400.000	2900.000	.000	.000	.000	.000
X1	.860	13.000	120.000	480.000	1550.000	1400.000	1500.000	.000	.000	.000
GR	420.000	.000	400.000	120.000	376.000	200.000	372.000	210.000	372.000	370.000
GR	376.000	380.000	400.000	480.000	405.000	550.000	400.000	600.000	395.000	800.000
GR	395.000	1700.000	400.000	2050.000	410.000	2400.000	.000	.000	.000	.000
X1	.940	14.000	75.000	255.000	400.000	400.000	400.000	.000	2.000	.000
GR	410.000	.000	408.000	50.000	383.000	75.000	380.000	110.000	367.000	120.000
GR	367.000	210.000	384.000	255.000	390.000	275.000	392.000	276.000	397.000	300.000
GR	400.000	680.000	398.000	800.000	400.000	1050.000	408.000	2000.000	.000	.000
X1	1.060	10.000	190.000	450.000	700.000	600.000	650.000	.000	.000	.000
GR	410.000	.000	400.000	75.000	390.000	190.000	377.000	240.000	373.000	250.000
GR	373.000	400.000	390.000	450.000	395.000	500.000	400.000	1050.000	407.000	1600.000
X1	1.270	14.000	550.000	820.000	1050.000	1050.000	1050.000	.000	.000	.000
GR	410.000	.000	400.000	280.000	390.000	550.000	380.000	590.000	378.000	610.000
GR	374.000	620.000	374.000	710.000	378.000	720.000	380.000	775.000	390.000	820.000
GR	400.000	920.000	405.000	1200.000	400.000	1500.000	408.000	3000.000	.000	.000
EJ	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

```
 SECNO     DEPTH     CWSEL     CRIWS     WSELK     EG        HV        HL        OLOSS    BANK ELEV
 Q         QLOB      QCH       QROB      ALOB      ACH       AROB      VOL       TWA     LEFT/RIGHT
 TIME      VLOB      VCH       VROB      XNL       XNCH      XNR       WTN       ELMIN        SSTA
 SLOPE     XLOBL     XLCH      XLOBR     ITRIAL    IDC       ICONT     CORAR     TOPWID       ENDST


*PROF 1


CCHV=     .100 CEHV=     .300
*SECNO  .080

3265 DIVIDED FLOW

      .08     24.00    392.00      .00    392.00    393.28      1.28       .00       .00    390.00
   53000.       65.    42754.   10181.       52.     4400.     1805.        0.        0.    390.00
      .00      1.26      9.72     5.64      .050      .045      .050      .000    368.00    248.00
  .001790        0.        0.       0.         0         0         0       .00    662.00   1168.00

FLOW DISTRIBUTION FOR SECNO=       .08          CWSEL=    392.00

STA=      248.      300.      530.      598.      820.      870.      890.      940.      960.      976.     1100.     1160.     1168.
  PER Q=        .1      80.7        .2        .1       3.0       4.0      10.2       1.1        .0        .1        .5        .0
   AREA=      52.0    4400.0      68.0      48.0     350.0     290.0     725.0     140.0      16.0      40.0     120.0       8.0
    VEL=       1.3       9.7       1.3       1.3       4.5       7.3       7.5       4.3       1.3       1.3       2.0       1.2
  DEPTH=       1.0      19.1       1.0        .2       7.0      14.5      14.5       7.0       1.0        .3       2.0       1.0

*SECNO  .210

3265 DIVIDED FLOW

      .21     24.41    393.41      .00       .00    394.70      1.30      1.42       .01    390.00
   53000.       19.    45029.    7952.       12.     4600.     2143.      117.       13.    390.00
      .02      1.67      9.79     3.71      .050      .045      .050      .000    369.00    143.19
  .001795      750.      800.     750.         2         0         0       .00    764.76   1000.43

FLOW DISTRIBUTION FOR SECNO=       .21          CWSEL=    393.41

STA=      143.      150.      390.      533.      680.      900.      980.     1000.
  PER Q=        .0      85.0        .8        .3      10.1       3.7        .1
   AREA=      11.6    4599.8     243.5      92.8    1299.2     472.4      34.8
    VEL=       1.7       9.8       1.8       1.8       4.1       4.1       1.8
  DEPTH=       1.7      19.2       1.7        .6       5.9       5.9       1.7

*SECNO  .340
```

```
 SECNO     DEPTH     CWSEL     CRIWS     WSELK     EG        HV        HL        OLOSS    BANK ELEV
 Q         QLOB      QCH       QROB      ALOB      ACH       AROB      VOL       TWA    LEFT/RIGHT
 TIME      VLOB      VCH       VROB      XNL       XNCH      XNR       WTN       ELMIN        SSTA
 SLOPE     XLOBL     XLCH      XLOBR     ITRIAL    IDC       ICONT     CORAR     TOPWID      ENDST


3265 DIVIDED FLOW


3301 HV CHANGED MORE THAN HVINS

      .34    24.36    394.36       .00       .00    396.33      1.97      1.42       .20    390.00
   53000.      57.    49689.     3254.       24.     4280.     1144.      212.       24.    390.00
      .04     2.39     11.61      2.85      .050      .045      .050      .000    370.00     39.11
  .002527     650.      670.      690.         2         0         0       .00    700.97   1797.42

FLOW DISTRIBUTION FOR SECNO=       .34          CWSEL=    394.36

STA=       39.       50.      275.      471.     1720.     1780.     1797.
  PER Q=       .1      93.8       2.0       2.0       2.0        .2
   AREA=     23.7    4280.0     426.9     417.4     261.3      37.9
    VEL=      2.4      11.6       2.5       2.5       4.0       2.5
  DEPTH=      2.2      19.0       2.2        .3       4.4       2.2

*SECNO  .550

3301 HV CHANGED MORE THAN HVINS

      .55    25.82    397.32       .00       .00    398.07       .75      1.62       .12    390.00
   53000.      47.    46971.     5982.       27.     6391.     3137.      401.       54.    390.00
      .09     1.74      7.35      1.91      .050      .045      .050      .000    371.50     12.68
  .000966    1100.     1100.     1100.         2         0         0       .00   1651.63   1664.31

FLOW DISTRIBUTION FOR SECNO=       .55          CWSEL=    397.32

STA=       13.       20.      350.      600.     1200.     1664.
  PER Q=       .1      88.6       6.0       4.3       1.0
   AREA=     26.8    6391.1    1205.4    1392.9     539.0
    VEL=      1.7       7.3       2.6       1.6       1.0
  DEPTH=      3.7      19.4       4.8       2.3       1.2

*SECNO  .860

3265 DIVIDED FLOW

      .86    26.81    398.81       .00       .00    399.40       .59      1.31       .02    400.00
   53000.       0.    44842.     8158.        0.     6740.     4238.      746.      108.    400.00
      .16      .00      6.65      1.92      .000      .045      .050      .000    372.00    123.94
  .000813    1550.     1500.     1400.         2         0         0       .00   1671.14   1967.27
```

```
  SECNO     DEPTH     CWSEL     CRIWS     WSELK     EG        HV        HL        OLOSS   BANK ELEV
  Q         QLOB      QCH       QROB      ALOB      ACH       AROB      VOL       TWA   LEFT/RIGHT
  TIME      VLOB      VCH       VROB      XNL       XNCH      XNR       WTN       ELMIN       SSTA
  SLOPE     XLOBL     XLCH      XLOBR     ITRIAL    IDC       ICONT     CORAR     TOPWID      ENDST

FLOW DISTRIBUTION FOR SECNO=       .86          CWSEL=    398.81

STA=      124.      480.      800.     1700.     1967.
  PER Q=      84.6        .7      13.4       1.3
   AREA=    6739.8     291.6    3436.4     510.2
    VEL=       6.7       1.3       2.1       1.3
  DEPTH=      19.2        .9       3.8       1.9

*SECNO  .940

3301 HV CHANGED MORE THAN HVINS

      .94     29.19    398.19       .00       .00    400.37      2.18       .49       .48    385.00
   53000.      327.    51514.     1159.       87.     4300.      231.      818.      117.    386.00
      .17      3.76     11.98      5.01      .050      .045      .050      .000    369.00     61.81
  .002051      400.      400.      400.         2         0         0       .00    234.32    296.13

FLOW DISTRIBUTION FOR SECNO=       .94          CWSEL=    398.19

STA=       62.       75.      255.      275.      276.      296.
  PER Q=        .6      97.2       2.0        .0        .2
   AREA=      87.0    4299.8     183.9       5.2      42.2
    VEL=       3.8      12.0       5.7       2.4       2.2
  DEPTH=       6.6      23.9       9.2       5.2       2.1

*SECNO 1.060

3301 HV CHANGED MORE THAN HVINS

     1.06     27.48    400.48       .00       .00    401.29       .81       .79       .14    390.00
   53000.     1647.    47154.     4199.      631.     6174.     2046.      917.      126.    390.00
      .19      2.61      7.64      2.05      .050      .045      .050      .000    373.00     71.42
  .000805      700.      650.      600.         2         0         0       .00   1016.14   1087.55

FLOW DISTRIBUTION FOR SECNO=      1.06          CWSEL=    400.48

STA=       71.      190.      450.      500.     1050.     1088.
  PER Q=       3.1      89.0       2.5       5.4        .0
   AREA=     630.8    6174.3     398.9    1637.9       9.0
    VEL=       2.6       7.6       3.4       1.7        .3
  DEPTH=       5.3      23.7       8.0       3.0        .2
```

```
 SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG       HV       HL       OLOSS   BANK ELEV
 Q        QLOB     QCH      QROB     ALOB     ACH      AROB     VOL      TWA    LEFT/RIGHT
 TIME     VLOB     VCH      VROB     XNL      XNCH     XNR      WTN      ELMIN      SSTA
 SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC      ICONT    CORAR    TOPWID     ENDST

*SECNO 1.270

3265 DIVIDED FLOW

   1.27    27.38   401.38      .00      .00   402.16      .79      .87      .00   390.00
 53000.    5149.   45761.    2090.    1749.    6042.     926.    1129.     151.   390.00
    .23     2.94     7.57     2.26     .050     .045     .050     .000   374.00   241.41
.000850    1050.    1050.    1050.        2        0        0      .00  1096.86  1758.41

FLOW DISTRIBUTION FOR SECNO=    1.27        CWSEL=   401.38

STA=     241.     280.     550.     820.     920.     997.    1500.    1758.
  PER Q=      .0      9.7     86.3      3.6       .1       .1       .2
   AREA=    26.6   1722.1   6042.1    637.8     53.2     57.0    178.1
    VEL=      .7      3.0      7.6      3.0       .7       .7       .7
  DEPTH=      .7      6.4     22.4      6.4       .7       .1       .7
```

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                                                                     THIS RUN EXECUTED 04JAN88  10:13:22
**************************************************
 1988 TRIAL VERSION OF HEC2
**************************************************

T1  FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2  Encroachment Method 4 with an 0.8 foot increase
T3      COW CREEK NEAR PALO CEDRO

   J1.2 = 3 reads the third field of ET; Method 4 with 0.8 foot increase
   J1.9 = 393 indicating a 1-foot higher starting water surface elevation

J1  ICHECK    INQ      NINV      IDIR      STRT      METRIC    HVINS     Q         WSEL      FQ

       0.        3.        0.        0.   .000000      .00        .0        0.   393.000     .000

   No Flow Distribution requested for this profile (J2.10 = 0)

J2  NPROF     IPLOT    PRFVS     XSECV     XSECH     FN        ALLDC     IBW       CHNIM     ITRACE

     2.000     .000    -1.000     .000      .000      .000      .000      .000      .000      .000
```

```
 SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG       HV       HL       OLOSS   BANK ELEV
 Q        QLOB     QCH      QROB     ALOB     ACH      AROB     VOL      TWA   LEFT/RIGHT
 TIME     VLOB     VCH      VROB     XNL      XNCH     XNR      WTN      ELMIN      SSTA
 SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC      ICONT    CORAR    TOPWID     ENDST

*PROF 2

CCHV=     .100 CEHV=     .300
*SECNO  .080
 2800 NAT Q1=  12528.17  WSEL=    392.00  ENC Q1=  12528.17  WSEL=    392.80  RATIO=   .0000
 NAT Q1=  13654.  RATIOS LOB,CH,ROB=   .0028   .7925   .2047  WSEL=    392.80

3265 DIVIDED FLOW

3470 ENCROACHMENT STATIONS=      300.0     916.8  TYPE=      4  TARGET=         .082
      .08     25.00   393.00      .00   392.00   394.35     1.35      .00      .00   390.00
   53000.        0.   45594.    7406.       0.    4630.    1418.       0.       0.   390.00
      .00       .00     9.85     5.22     .000     .045     .050     .000   368.00   300.00
  .001746        0.       0.       0.        0        0        0      .00   500.84   916.84

*SECNO  .210
 2800 NAT Q1=  12508.18  WSEL=    393.41  ENC Q1=  12508.18  WSEL=    394.21  RATIO=   .0000
 NAT Q1=  13785.  RATIOS LOB,CH,ROB=   .0006   .8253   .1742  WSEL=    394.21

3265 DIVIDED FLOW

3470 ENCROACHMENT STATIONS=      150.0     835.2  TYPE=      4  TARGET=         .093
      .21     25.36   394.36      .00   393.41   395.77     1.40     1.40      .01   390.00
   53000.        0.   47982.    5018.       0.    4830.    1503.     112.      10.   390.00
      .02       .00     9.94     3.34     .000     .045     .050     .000   369.00   150.00
  .001774      750.     800.     750.        2        0        0      .00   648.29   835.24

*SECNO  .340
 2800 NAT Q1=  10542.66  WSEL=    394.36  ENC Q1=  10586.18  WSEL=    395.16  RATIO=  -.0041
 NAT Q1=  11594.  RATIOS LOB,CH,ROB=   .0015   .9131   .0854  WSEL=    395.16

3301 HV CHANGED MORE THAN HVINS

3470 ENCROACHMENT STATIONS=       50.0     275.0  TYPE=      4  TARGET=         .087
      .34     25.26   395.26      .00   394.36   397.43     2.17     1.43      .23   390.00
   53000.        0.   53000.       0.       0.    4483.       0.     196.      17.   390.00
      .04       .00    11.82      .00     .000     .045     .000     .000   370.00    50.00
  .002614      650.     670.     690.        2        0        0      .00   225.00   275.00
```

```
 SECNO     DEPTH     CWSEL     CRIWS     WSELK     EG        HV        HL        OLOSS   BANK ELEV
 Q         QLOB      QCH       QROB      ALOB      ACH       AROB      VOL       TWA   LEFT/RIGHT
 TIME      VLOB      VCH       VROB      XNL       XNCH      XNR       WTN       ELMIN       SSTA
 SLOPE     XLOBL     XLCH      XLOBR     ITRIAL    IDC       ICONT     CORAR     TOPWID      ENDST


*SECNO  .550
 2800 NAT Q1=  17055.23  WSEL=    397.32  ENC Q1=  17055.23  WSEL=    398.12  RATIO=   .0000
 NAT Q1=  19085.  RATIOS LOB,CH,ROB=   .0010    .8472    .1518  WSEL=    398.12

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=       20.0      473.4  TYPE=      4  TARGET=          .106
      .55     26.87    398.37       .00    397.32    399.19       .82      1.63       .13   390.00
   53000.        0.    50099.     2901.        0.     6737.      880.      348.       26.    390.00
      .08       .00      7.44      3.30      .000      .045      .050      .000    371.50    20.00
  .000952     1100.     1100.     1100.         2         0         0       .00    453.36   473.36

*SECNO  .860
 2800 NAT Q1=  18592.46  WSEL=    398.81  ENC Q1=  18592.46  WSEL=    399.61  RATIO=   .0000
 NAT Q1=  20669.  RATIOS LOB,CH,ROB=   .0000    .8054    .1946  WSEL=    399.61

3265 DIVIDED FLOW


3470 ENCROACHMENT STATIONS=      120.0     1257.7  TYPE=      4  TARGET=          .100
      .86     27.88    399.88       .00    398.81    400.49       .61      1.27       .02   400.00
   53000.        0.    47038.     5962.        0.     7115.     2707.      644.       50.    400.00
      .15       .00      6.61      2.20      .000      .045      .050      .000    372.00   120.41
  .000769     1550.     1500.     1400.         2         0         0       .00   1011.80  1257.68

*SECNO  .940
 2800 NAT Q1=  11704.20  WSEL=    398.19  ENC Q1=  12015.00  WSEL=    398.99  RATIO=  -.0266
 NAT Q1=  12403.  RATIOS LOB,CH,ROB=   .0068    .9687    .0245  WSEL=    398.99

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=       75.0      255.0  TYPE=      4  TARGET=          .031
      .94     30.28    399.28       .00    398.19    401.44      2.16       .49       .46   385.00
   53000.        0.    53000.        0.        0.     4497.        0.      710.       55.    386.00
      .16       .00     11.79       .00      .000      .045      .000      .000    369.00    75.00
```

```
 SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG       HV       HL       OLOSS   BANK ELEV
 Q        QLOB     QCH      QROB     ALOB     ACH      AROB     VOL      TWA   LEFT/RIGHT
 TIME     VLOB     VCH      VROB     XNL      XNCH     XNR      WTN      ELMIN      SSTA
 SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC      ICONT    CORAR    TOPWID     ENDST


*SECNO 1.060
 2800 NAT Q1=  18684.73  WSEL=    400.48  ENC Q1=  18684.73  WSEL=    401.28  RATIO=   .0000
 NAT Q1=  20360.  RATIOS LOB,CH,ROB=   .0358   .8628   .1014  WSEL=    401.28

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=      190.0     605.8  TYPE=      4  TARGET=         .082
     1.06    28.52   401.52      .00   400.48   402.39      .87      .82      .13   390.00
   53000.       0.   49603.    3397.       0.    6447.    1091.     799.      60.   390.00
      .18      .00     7.69     3.11     .000     .045     .050     .000   373.00   190.00
  .000816     700.     650.     600.        2        0        0      .00   415.83   605.83

*SECNO 1.270
 2800 NAT Q1=  18173.66  WSEL=    401.38  ENC Q1=  18173.66  WSEL=    402.18  RATIO=   .0000
 NAT Q1=  19816.  RATIOS LOB,CH,ROB=   .1091   .8395   .0513  WSEL=    402.18

3470 ENCROACHMENT STATIONS=      454.0     833.1  TYPE=      4  TARGET=         .083
     1.27    28.39   402.39      .00   401.38   403.25      .86      .86      .00   390.00
   53000.    3966.   48583.     451.    1019.    6316.     154.     981.      69.   390.00
      .22     3.89     7.69     2.93     .050     .045     .050     .000   374.00   453.99
  .000827    1050.    1050.    1050.        2        0        0      .00   379.13   833.13
```

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                                                             THIS RUN EXECUTED 04JAN88  10:13:36
**************************************************
 1988 TRIAL VERSION OF HEC2
**************************************************

T1  FLOODWAY DETERMINATION EXAMPLE - First Trial -
T2  Encroachment Method 4 with a 1.0 foot increase
T3      COW CREEK NEAR PALO CEDRO

   J1.2 = 4 reads the fourth field of ET; Method 4 with a 1.0 foot rise
   J1.9 = 393 indicating a 1-foot higher starting water surface elevation

J1  ICHECK    INQ      NINV      IDIR      STRT      METRIC    HVINS     Q         WSEL      FQ

       0.        4.        0.        0.   .000000       .00        .0        0.   393.000      .000

   J2.1 = 15 requesting the Floodway Summary Tables specified on J3

J2  NPROF     IPLOT    PRFVS     XSECV     XSECH     FN        ALLDC     IBW       CHNIM     ITRACE

     15.000      .000    -1.000      .000      .000      .000      .000      .000      .000      .000
```

```
  SECNO     DEPTH     CWSEL     CRIWS     WSELK     EG        HV        HL        OLOSS   BANK ELEV
  Q         QLOB      QCH       QROB      ALOB      ACH       AROB      VOL       TWA    LEFT/RIGHT
  TIME      VLOB      VCH       VROB      XNL       XNCH      XNR       WTN       ELMIN       SSTA
  SLOPE     XLOBL     XLCH      XLOBR     ITRIAL    IDC       ICONT     CORAR     TOPWID      ENDST


*PROF 3


CCHV=     .100 CEHV=     .300
*SECNO  .080
 2800 NAT Q1=  12528.17  WSEL=    392.00  ENC Q1=  12528.17  WSEL=    393.00  RATIO=   .0000
 NAT Q1=  13951.  RATIOS LOB,CH,ROB=    .0033    .7886    .2081  WSEL=    393.00

3265 DIVIDED FLOW


3470 ENCROACHMENT STATIONS=      300.0     906.9  TYPE=      4  TARGET=         .102
      .08     25.00    393.00       .00    392.00    394.43      1.43       .00       .00    390.00
   53000.        0.    46604.     6396.        0.     4630.     1261.        0.        0.    390.00
      .00       .00     10.07      5.07      .000      .045      .050      .000    368.00    300.00
  .001824        0.        0.        0.         0         0         0       .00    490.91    906.91

*SECNO  .210
 2800 NAT Q1=  12508.18  WSEL=    393.41  ENC Q1=  12508.18  WSEL=    394.41  RATIO=   .0000
 NAT Q1=  14120.  RATIOS LOB,CH,ROB=    .0006    .8192    .1802  WSEL=    394.41

3265 DIVIDED FLOW


3470 ENCROACHMENT STATIONS=      150.0     802.6  TYPE=      4  TARGET=         .114
      .21     25.42    394.42       .00    393.41    395.90      1.48      1.45       .01    390.00
   53000.        0.    48957.     4043.        0.     4843.     1280.      109.       10.    390.00
      .02       .00     10.11      3.16      .000      .045      .050      .000    369.00    150.00
  .001829      750.      800.      750.         2         0         0       .00    619.03    802.64

*SECNO  .340
 2800 NAT Q1=  10542.66  WSEL=    394.36  ENC Q1=  10764.82  WSEL=    395.36  RATIO=  -.0211
 NAT Q1=  11922.  RATIOS LOB,CH,ROB=    .0016    .9030    .0954  WSEL=    395.36

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=       50.0     275.0  TYPE=      4  TARGET=         .097
```

```
  SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG       HV       HL       OLOSS   BANK ELEV
  Q        QLOB     QCH      QROB     ALOB     ACH      AROB     VOL      TWA    LEFT/RIGHT
  TIME     VLOB     VCH      VROB     XNL      XNCH     XNR      WTN      ELMIN       SSTA
  SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC      ICONT    CORAR    TOPWID      ENDST


*SECNO  .550
 2800 NAT Q1=  17055.23  WSEL=    397.32  ENC Q1=  17055.23  WSEL=    398.32  RATIO=   .0000
 NAT Q1=  19634.  RATIOS LOB,CH,ROB=   .0011   .8371   .1618  WSEL=    398.32

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=       20.0     424.6  TYPE=      4  TARGET=         .131
      .55    26.95   398.45      .00   397.32   399.31      .86     1.64      .13   390.00
   53000.       0.   51040.    1960.       0.    6763.     574.     341.      24.   390.00
      .08      .00     7.55     3.41     .000     .045     .050     .000   371.50    20.00
  .000977    1100.    1100.    1100.        2        0        0      .00   404.56   424.56

*SECNO  .860
 2800 NAT Q1=  18592.46  WSEL=    398.81  ENC Q1=  18592.46  WSEL=    399.81  RATIO=   .0000
 NAT Q1=  21228.  RATIOS LOB,CH,ROB=   .0000   .7953   .2047  WSEL=    399.81

3265 DIVIDED FLOW


3470 ENCROACHMENT STATIONS=      120.0    1161.9  TYPE=      4  TARGET=         .124
      .86    28.00   400.00      .00   398.81   400.63      .63     1.30      .02   400.00
   53000.       0.   47877.    5123.       0.    7159.    2308.     627.      46.   400.00
      .15      .00     6.69     2.22     .000     .045     .050     .000   372.00   120.01
  .000783    1550.    1500.    1400.        2        0        0      .00   921.81  1161.94

*SECNO  .940
 2800 NAT Q1=  11704.20  WSEL=    398.19  ENC Q1=  12177.70  WSEL=    399.19  RATIO=  -.0405
 NAT Q1=  12582.  RATIOS LOB,CH,ROB=   .0070   .9678   .0252  WSEL=    399.19

3301 HV CHANGED MORE THAN HVINS


3470 ENCROACHMENT STATIONS=       75.0     255.0  TYPE=      4  TARGET=         .032
      .94    30.44   399.44      .00   398.19   401.57     2.13      .49      .45   385.00
   53000.       0.   53000.       0.       0.    4525.       0.     691.      52.   386.00
      .16      .00    11.71      .00     .000     .045     .000     .000   369.00    75.00
```

```
 SECNO    DEPTH    CWSEL    CRIWS    WSELK    EG       HV       HL       OLOSS   BANK ELEV
 Q        QLOB     QCH      QROB     ALOB     ACH      AROB     VOL      TWA   LEFT/RIGHT
 TIME     VLOB     VCH      VROB     XNL      XNCH     XNR      WTN      ELMIN       SSTA
 SLOPE    XLOBL    XLCH     XLOBR    ITRIAL   IDC      ICONT    CORAR    TOPWID      ENDST

*SECNO 1.060
 2800 NAT Q1=  18684.73  WSEL=    400.48  ENC Q1=  18684.73  WSEL=    401.48  RATIO=   .0000
 NAT Q1=  20801.  RATIOS LOB,CH,ROB=   .0370   .8560   .1070  WSEL=    401.48

3301 HV CHANGED MORE THAN HVINS

3470 ENCROACHMENT STATIONS=      190.0     550.9  TYPE=      4  TARGET=         .102
     1.06     28.62   401.62      .00   400.48   402.52      .90      .83      .12   390.00
   53000.        0.   50409.    2591.       0.    6471.     781.     778.      55.   390.00
      .18       .00     7.79     3.32     .000     .045     .050     .000   373.00   190.00
  .000832      700.     650.     600.        2        0        0      .00   360.90   550.90

*SECNO 1.270
 2800 NAT Q1=  18173.66  WSEL=    401.38  ENC Q1=  18173.66  WSEL=    402.38  RATIO=   .0000
 NAT Q1=  20259.  RATIOS LOB,CH,ROB=   .1120   .8330   .0550  WSEL=    402.38

3470 ENCROACHMENT STATIONS=      467.7     824.5  TYPE=      4  TARGET=         .103
     1.27     28.51   402.51      .00   401.38   403.40      .89      .88      .00   390.00
   53000.     3574.   49321.     105.     904.    6348.      55.     954.      64.   390.00
      .22      3.95     7.77     1.91     .050     .045     .050     .000   374.00   467.72
  .000838     1050.    1050.    1050.        2        0        0      .00   356.74   824.45
```

THIS RUN EXECUTED 04JAN88 10:13:50

****
1988 TRIAL VERSION OF HEC2
****

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

COW CREEK NEAR PALO CEDR

SUMMARY PRINTOUT TABLE 110

SECNO	CWSEL	DIFKWS	EG	TOPWID	QLOB	QCH	QROB	PERENC	STENCL	STCHL	STCHR	STENCR
.080	392.00	.00	393.28	662.00	65.35	42754.07	10180.58	.00	.00	300.00	530.00	.00
.080	393.00	1.00	394.36	500.84	.00	45593.72	7406.28	.08	300.00	300.00	530.00	916.84
.080	393.00	1.00	394.43	490.91	.00	46604.09	6395.91	.10	300.00	300.00	530.00	906.91
.210	393.40	.00	394.70	764.76	19.33	45029.04	7951.63	.00	.00	150.00	390.00	.00
.210	394.36	.96	395.77	648.29	.00	47982.34	5017.65	.09	150.00	150.00	390.00	835.24
.210	394.42	1.01	395.90	619.03	.00	48957.30	4042.70	.11	150.00	150.00	390.00	802.64
.340	394.36	.00	396.33	700.97	56.66	49689.46	3253.88	.00	.00	50.00	275.00	.00
.340	395.26	.90	397.43	225.00	.00	53000.00	.00	.09	50.00	50.00	275.00	275.00
.340	395.40	1.04	397.54	225.00	.00	53000.00	.00	.10	50.00	50.00	275.00	275.00
.550	397.32	.00	398.07	1651.63	46.67	46970.93	5982.41	.00	.00	20.00	350.00	.00
.550	398.37	1.05	399.19	453.36	.00	50098.70	2901.29	.11	20.00	20.00	350.00	473.36
.550	398.45	1.13	399.31	404.56	.00	51040.14	1959.86	.13	20.00	20.00	350.00	424.57
.860	398.81	.00	399.40	1671.14	.00	44842.41	8157.59	.00	.00	120.00	480.00	.00
.860	399.88	1.07	400.49	1011.80	.00	47037.81	5962.19	.10	120.00	120.00	480.00	1257.69
.860	400.00	1.19	400.63	921.82	.00	47876.91	5123.08	.12	120.00	120.00	480.00	1161.94
.940	398.19	.00	400.37	234.32	327.01	51513.74	1159.25	.00	.00	75.00	255.00	.00
.940	399.28	1.09	401.44	180.00	.00	53000.00	.00	.03	75.00	75.00	255.00	255.00
.940	399.44	1.25	401.57	180.00	.00	53000.00	.00	.03	75.00	75.00	255.00	255.00
1.060	400.48	.00	401.29	1016.14	1646.55	47154.19	4199.26	.00	.00	190.00	450.00	.00
1.060	401.52	1.04	402.39	415.83	.00	49603.12	3396.88	.08	190.00	190.00	450.00	605.83
1.060	401.62	1.14	402.52	360.90	.00	50408.72	2591.28	.10	190.00	190.00	450.00	550.90
1.270	401.38	.00	402.16	1096.86	5149.36	45760.67	2089.97	.00	.00	550.00	820.00	.00
1.270	402.39	1.02	403.26	379.13	3965.95	48582.71	451.34	.08	453.99	550.00	820.00	833.13
1.270	402.51	1.13	403.40	356.74	3574.09	49321.34	104.56	.10	467.72	550.00	820.00	824.46

SUMMARY OF ERRORS AND SPECIAL NOTES

FLOODWAY DATA, COW CREEK NEAR PALO CEDR
PROFILE NO. 2

STATION	FLOODWAY WIDTH	FLOODWAY SECTION AREA	FLOODWAY MEAN VELOCITY	WATER SURFACE ELEVATION WITH FLOODWAY	WATER SURFACE ELEVATION WITHOUT FLOODWAY	WATER SURFACE ELEVATION DIFFERENCE
.080	617.	6048.	8.8	393.0	392.0	1.0
.210	685.	6333.	8.4	394.4	393.4	1.0
.340	225.	4483.	11.8	395.3	394.4	.9
.550	453.	7618.	7.0	398.4	397.3	1.1
.860	1137.	9823.	5.4	399.9	398.8	1.1
.940	180.	4497.	11.8	399.3	398.2	1.1
1.060	416.	7537.	7.0	401.5	400.5	1.0
1.270	379.	7490.	7.1	402.4	401.4	1.0

FLOODWAY DATA, COW CREEK NEAR PALO CEDR
PROFILE NO. 3

STATION	FLOODWAY WIDTH	FLOODWAY SECTION AREA	FLOODWAY MEAN VELOCITY	WATER SURFACE ELEVATION WITH FLOODWAY	WATER SURFACE ELEVATION WITHOUT FLOODWAY	WATER SURFACE ELEVATION DIFFERENCE
.080	607.	5891.	9.0	393.0	392.0	1.0
.210	653.	6123.	8.7	394.4	393.4	1.0
.340	225.	4514.	11.7	395.4	394.4	1.0
.550	405.	7337.	7.2	398.4	397.3	1.1
.860	1042.	9467.	5.6	400.0	398.8	1.2
.940	180.	4525.	11.7	399.5	398.2	1.3
1.060	361.	7253.	7.3	401.6	400.5	1.1
1.270	357.	7307.	7.3	402.5	401.4	1.1

THIS RUN EXECUTED 04JAN88 10:14:19

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**************************************************
 1988 TRIAL VERSION OF HEC2
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Appendix C HEC-2 FLOODWAY EXAMPLE - SECOND TRIAL

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*******************************************************          **************************************
*  WATER SURFACE PROFILES                             *          *  U.S. ARMY CORPS OF ENGINEERS      *
*  VERSION OF NOVEMBER 1976                           *          *  THE HYDROLOGIC ENGINEERING CENTER *
*  EXPERIMENTAL VERSION OF 1988 RELEASE               *          *  609 SECOND STREET, SUITE D        *
*                                                     *          *  DAVIS, CALIFORNIA 95616           *
*  RUN DATE     1/ 4/88   TIME   10:39:23             *          *  (916) 756-1104 (FTS) 460-1748     *
*******************************************************          **************************************


                                        X     X  XXXXXXX   XXXXX              XXXXX
                                        X     X  X        X     X            X     X
                                        X     X  X        X                        X
                                        XXXXXXX  XXXX     X          XXXXX    XXXXX
                                        X     X  X        X                  X
                                        X     X  X        X     X            X
                                        X     X  XXXXXXX   XXXXX             XXXXXXX
END OF BANNER
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                                                                THIS RUN EXECUTED 04JAN88  10:39:24
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 1988 TRIAL VERSION OF HEC2
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T1  FLOODWAY DETERMINATION EXAMPLE - Second Trial -
T2  First Profile Existing Condition; Second Method 4, & Third Method 1
T3      COW CREEK NEAR PALO CEDRO

J1  ICHECK     INQ       NINV      IDIR      STRT      METRIC    HVINS      Q        WSEL      FQ

       0.        2.        0.        0.   .000000       .00        .0       0.   392.000     .000

J2  NPROF     IPLOT     PRFVS     XSECV     XSECH      FN       ALLDC      IBW      CHNIM    ITRACE

     1.000      .000    -1.000      .000      .000      .000      .000      .000      .000    15.000

J3  VARIABLE CODES FOR SUMMARY PRINTOUT

   110.000   200.000      .000      .000      .000      .000      .000      .000      .000      .000


NC       .050      .050      .045      .100      .300      .000      .000      .000      .000      .000
QT      3.000 53000.000 53000.000 53000.000      .000      .000      .000      .000      .000      .000

     ET.3 requests Method 4 with 0.9 foot rise
     ET.4 indicates Method 1 with Encroachment stations in fields 5 and 6
          Selected stations (300 & 920) were based on the First Trial results.

ET       .000      .000     9.400     5.100   300.000   920.000      .000      .000      .000      .000
X1       .080    21.000   300.000   530.000      .000      .000      .000      .000      .000      .000
GR    410.000      .000   400.000    40.000   390.000   300.000   380.000   330.000   372.000   350.000
GR    368.000   360.000   368.000   480.000   372.000   490.000   380.000   520.000   390.000   530.000
GR    395.000   700.000   390.000   820.000   380.000   870.000   375.000   890.000   380.000   940.000
GR    390.000   960.000   395.000  1000.000   390.000  1100.000   390.000  1160.000   400.000  1200.000
GR    410.000  1500.000      .000      .000      .000      .000      .000      .000      .000      .000

     ET.3 changes the Method 4 target to 0.8 foot rise
     ET.4 defines the Encroachment stations for the next cross section.
          Method 1 only applies to the following cross section; therefore, an
          ET is required before each corss section when using Method 1.
                                                 STENCL    STENCR

ET       .000      .000     8.400     5.100   150.000   820.000      .000      .000      .000      .000
X1       .210    18.000   150.000   390.000   750.000   750.000   800.000      .000      .000      .000
GR    410.000      .000   400.000   130.000   390.000   150.000   380.000   175.000   373.000   200.000
GR    369.000   210.000   369.000   300.000   373.000   320.000   380.000   380.000   390.000   390.000
GR    395.000   600.000   390.000   680.000   385.000   900.000   390.000   980.000   400.000  1040.000
GR    403.000  1100.000   395.000  1400.000   400.000  1600.000      .000      .000      .000      .000
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					STENCL	STENCR				
ET	.000	.000	6.400	5.100	50.000	350.000	.000	.000	.000	.000
X1	.340	16.000	50.000	275.000	650.000	690.000	670.000	.000	.000	.000
GR	410.000	.000	390.000	50.000	373.500	100.000	370.000	110.000	370.000	210.000
GR	373.500	220.000	380.000	250.000	390.000	275.000	395.000	500.000	395.000	1500.000
GR	390.000	1720.000	390.000	1780.000	400.000	1820.000	403.000	2200.000	398.000	2900.000
GR	400.000	3300.000	.000	.000	.000	.000	.000	.000	.000	.000

ET.3 is blank, indicating no change from previous value of 6.4. All the
Methods, except 1, apply continuously, unles changed by a new entry.

			BLANK		STENCL	STENCR				
ET	.000	.000	.000	5.100	20.000	500.000	.000	.000	.000	.000
X1	.550	13.000	20.000	350.000	1100.000	1100.000	1100.000	.000	.000	.000
GR	410.000	.000	390.000	20.000	374.500	90.000	371.500	100.000	371.500	200.000
GR	374.500	210.000	380.000	280.000	390.000	350.000	395.000	600.000	395.000	1200.000
GR	400.000	2200.000	405.000	2350.000	400.000	2900.000	.000	.000	.000	.000
ET	.000	.000	.000	5.100	120.000	1120.000	.000	.000	.000	.000
X1	.860	13.000	120.000	480.000	1550.000	1400.000	1500.000	.000	.000	.000
GR	420.000	.000	400.000	120.000	376.000	200.000	372.000	210.000	372.000	370.000
GR	376.000	380.000	400.000	480.000	405.000	550.000	400.000	600.000	395.000	800.000
GR	395.000	1700.000	400.000	2050.000	410.000	2400.000	.000	.000	.000	.000
ET	.000	.000	.000	5.100	75.000	260.000	.000	.000	.000	.000
X1	.940	14.000	75.000	255.000	400.000	400.000	400.000	.000	2.000	.000
GR	410.000	.000	408.000	50.000	383.000	75.000	380.000	110.000	367.000	120.000
GR	367.000	210.000	384.000	255.000	390.000	275.000	392.000	276.000	397.000	300.000
GR	400.000	680.000	398.000	800.000	400.000	1050.000	408.000	2000.000	.000	.000
ET	.000	.000	8.400	5.100	190.000	590.000	.000	.000	.000	.000
X1	1.060	10.000	190.000	450.000	700.000	600.000	650.000	.000	.000	.000
GR	410.000	.000	400.000	75.000	390.000	190.000	377.000	240.000	373.000	250.000
GR	373.000	400.000	390.000	450.000	395.000	500.000	400.000	1050.000	407.000	1600.000
ET	.000	.000	.000	5.100	450.000	830.000	.000	.000	.000	.000
X1	1.270	14.000	550.000	820.000	1050.000	1050.000	1050.000	.000	.000	.000
GR	410.000	.000	400.000	280.000	390.000	550.000	380.000	590.000	378.000	610.000
GR	374.000	620.000	374.000	710.000	378.000	720.000	380.000	775.000	390.000	820.000
GR	400.000	920.000	405.000	1200.000	400.000	1500.000	408.000	3000.000	.000	.000
EJ	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

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1988 TRIAL VERSION OF HEC2
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NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

COW CREEK NEAR PALO CEDR

SUMMARY PRINTOUT TABLE 110

SECNO	CWSEL	DIFKWS	EG	TOPWID	QLOB	QCH	QROB	PERENC	STENCL	STCHL	STCHR	STENCR
.080	392.00	.00	393.28	662.00	65.35	42754.07	10180.58	.00	.00	300.00	530.00	.00
.080	393.00	1.00	394.39	495.88	.00	46091.56	6908.44	.09	300.00	300.00	530.00	911.89
.080	393.00	1.00	394.33	504.00	.00	45286.80	7713.20	620.00	300.00	300.00	530.00	920.00
.210	393.40	.00	394.70	764.76	19.33	45029.04	7951.63	.00	.00	150.00	390.00	.00
.210	394.41	1.00	395.80	650.76	.00	47930.48	5069.51	.09	150.00	150.00	390.00	835.24
.210	394.32	.92	395.78	630.59	.00	48532.24	4467.75	670.00	150.00	150.00	390.00	820.00
.340	394.36	.00	396.33	700.97	56.66	49689.46	3253.88	.00	.00	50.00	275.00	.00
.340	395.31	.96	397.38	264.01	.00	52251.02	748.98	.07	50.00	50.00	275.00	314.01
.340	395.33	.98	397.34	300.00	.00	51706.63	1293.37	300.00	50.00	50.00	275.00	350.00
.550	397.32	.00	398.07	1651.63	46.67	46970.93	5982.41	.00	.00	20.00	350.00	.00
.550	398.30	.97	399.08	535.98	.00	49116.97	3883.03	.08	20.00	20.00	350.00	555.98
.550	398.22	.90	399.04	480.00	.00	49762.91	3237.10	480.00	20.00	20.00	350.00	500.00
.860	398.81	.00	399.40	1671.14	.00	44842.41	8157.59	.00	.00	120.00	480.00	.00
.860	399.75	.94	400.34	1114.26	.00	46174.23	6825.78	.08	120.00	120.00	480.00	1366.08
.860	399.71	.90	400.40	866.64	.00	48690.20	4309.80	1000.00	120.00	120.00	480.00	1120.00
.940	398.19	.00	400.37	234.32	327.01	51513.74	1159.25	.00	.00	75.00	255.00	.00
.940	399.12	.93	401.30	180.00	.00	53000.00	.00	.03	75.00	75.00	255.00	255.00
.940	399.19	1.00	401.34	185.00	.00	52799.65	200.35	185.00	75.00	75.00	255.00	260.00
1.060	400.48	.00	401.29	1016.14	1646.55	47154.19	4199.26	.00	.00	190.00	450.00	.00
1.060	401.39	.92	402.27	415.83	.00	49670.80	3329.20	.08	190.00	190.00	450.00	605.83
1.060	401.38	.91	402.28	400.00	.00	49895.74	3104.26	400.00	190.00	190.00	450.00	590.00
1.270	401.38	.00	402.16	1096.86	5149.36	45760.67	2089.97	.00	.00	550.00	820.00	.00
1.270	402.28	.91	403.15	379.13	3933.56	48616.97	449.47	.08	453.99	550.00	820.00	833.13

SUMMARY OF ERRORS AND SPECIAL NOTES

FLOODWAY DATA, COW CREEK NEAR PALO CEDR
PROFILE NO. 2

STATION	------- FLOODWAY ------- WIDTH	SECTION AREA	MEAN VELOCITY	WATER SURFACE ELEVATION WITH FLOODWAY	WITHOUT FLOODWAY	DIFFERENCE
.080	612.	5971.	8.9	393.0	392.0	1.0
.210	685.	6360.	8.3	394.4	393.4	1.0
.340	264.	4686.	11.3	395.4	394.4	1.0
.550	536.	7996.	6.6	398.3	397.3	1.0
.860	1245.	10212.	5.2	399.7	398.8	.9
.940	180.	4467.	11.9	399.1	398.2	.9
1.060	416.	7483.	7.1	401.4	400.5	.9
1.270	379.	7448.	7.1	402.3	401.4	.9

FLOODWAY DATA, COW CREEK NEAR PALO CEDR
PROFILE NO. 3

	------- FLOODWAY -------			WATER SURFACE ELEVATION		
STATION	WIDTH	SECTION AREA	MEAN VELOCITY	WITH FLOODWAY	WITHOUT FLOODWAY	DIFFERENCE
.080	620.	6096.	8.7	393.0	392.0	1.0
.210	670.	6188.	8.6	394.3	393.4	.9
.340	300.	4837.	11.0	395.4	394.4	1.0
.550	480.	7697.	6.9	398.2	397.3	.9
.860	999.	9014.	5.9	399.7	398.8	.9
.940	185.	4542.	11.7	399.2	398.2	1.0
1.060	400.	7393.	7.2	401.4	400.5	.9
1.270	380.	7451.	7.1	402.3	401.4	.9